The cone penetration test in unsaturated sands

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THE CONE PENETRATION TEST IN UNSATURATED SANDS

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BEng, MSc

A thesis submitted in fulfilment of the requirements for the degree of
DOCTOR OF PHILOSOPHY

School of Civil and Environmental Engineering
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The cone penetration test (CPT) is widely used for in situ characterization of saturated or dry soils based on correlations developed for saturated/dry conditions. However, no methods currently exist for interpretation of CPT data in unsaturated soils. This study advances both experimental and theoretical bases for interpretation of CPT data in unsaturated soils.

In particular, a calibration chamber has been designed and constructed to perform laboratory controlled cone penetration tests in unsaturated soils. The chamber allows the independent application of lateral and vertical pressures to an unsaturated soil specimen while suction is controlled using the axis translation technique. A unique feature of the chamber includes the specimen formation system, enabling dry pluviation, static or dynamic compaction to be used. The results of cone penetration tests conducted on saturated and unsaturated sand specimens are presented and the significant contribution of suction to cone penetration resistance is highlighted.

In addition, a novel solution procedure for the problem of cavity expansion in finite soil media has been developed which enables evaluation of chamber boundary effects and the extension of CPT results from chambers to free field conditions. It is possible to use hardening/softening elastic-plastic constitutive models formulated in the critical state framework in the analysis for the first time.

A simple method is then presented which enables either: i) the contribution of suction to the effective stress ($\gamma_s$) in an unsaturated sand to be estimated when results of CPTs are available for both saturated and unsaturated ground conditions; ii) an equivalent saturated cone penetration resistance to be determined from a value measured for unsaturated conditions, provided that $\gamma_s$ is known for the unsaturated conditions, permitting the use of established charts to estimate strength and stiffness parameters.

Established CPT correlations for sands are extended to account for influences of suction. The extended correlations enable direct characterization of an unsaturated sand from CPT results provided the contribution of suction to effective stress ($\gamma_s$) is known. It is shown that failing to account for $\gamma_s$ in the mean (or vertical) effective stress when interpreting CPT results may result in significant and non-conservative misrepresentations in estimated sand parameters.

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ABSTRACT

The cone penetration test (CPT) is widely used for in situ characterization of saturated or dry soils based on correlations developed for saturated/dry conditions. However, no methods currently exist for interpretation of CPT data in unsaturated soils. This study advances both experimental and theoretical bases for interpretation of CPT data in unsaturated soils.

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CHAPTER 1

INTRODUCTION

1.1 OVERVIEW AND PROBLEM STATEMENT

The engineering behaviour of unsaturated soils is complex and influenced by many factors including externally applied stresses, soil type, structure, density, and suction. Most notably, suction increases the soil’s strength and stiffness. Upon wetting, however, an unsaturated soil may weaken and exhibit volumetric collapse. Unsaturated soils are wide spread and need to be dealt with in many engineering problems including foundations, fills, embankment dams, pavements and airfields as well as slopes.

In recent years there have been major advances in unsaturated soil mechanics research (Alonso et al., 1990; Kogho et al., 1993; Wheeler and Sivakumar, 1995; Bolzon et al., 1996; Loret and Khalili, 2000, 2002; Gallipoli et al., 2003a; Wheeler et al., 2003; Sheng et al., 2004; Russell and Khalili, 2006a; Santagiuliana and Schrefler, 2006; Khalili et al., 2008, among others). Even so, the characterization of unsaturated soils requires expensive and time consuming site investigations involving soil borings and
undisturbed sampling for laboratory testing to obtain even the most basic information on the suction-dependant strength and stiffness.

Performing the cone penetration test (CPT) in unsaturated soils, a widely used in situ test, may enable less costly and more rapid characterization of the soil properties. The CPT is one of the most versatile tools for soil exploration and it permits continuous soundings with good repeatability. However, as is the case with other in situ tests, the cone penetrometer does not provide a direct measurement of any particular soil property. Rather, a load response to an imposed deformation on the soil as the cone penetrates is measured.

Indeed, the evaluation of saturated soil engineering properties from cone penetration test results has been an area of intense practical interest and many correlations have been developed linking cone penetration resistance and sleeve friction to relative density, in situ state and shear strength (e.g. Schmertmann, 1978; Villet and Mitchell, 1981; Baldi et al., 1982; Robertson and Campanella, 1983a, 1983b; Keaveny and Mitchell, 1986; Aas et al., 1986; Konrad and Law, 1987; Mayne, 1991; Jamiolkowski et al., 2001).

However, there is an absence of correlations for the CPT that take into account the effects of unsaturation. Engineers are left to interpret CPTs performed in unsaturated soils using correlations developed for saturated soils leading to unknown misrepresentations in estimated soil properties.

There is clear (albeit limited) evidence that suction significantly affects CPT results conducted in unsaturated soils (Hryciw and Dowding, 1987; Lehane et al., 2004; Russell and Khalili, 2006b). The need is for more extensive theoretical as well as experimental studies focusing on interpretation of CPT data in unsaturated soils and is thus the focus of this research.

Calibration chamber testing plays an important role in establishing experimental correlations for the CPT (Ghionna and Jamiolkowski, 1991). Many calibration chambers in different sizes and designs have been developed in the past to conduct
laboratory controlled CPTs in dry or saturated soils (e.g. Holden, 1971; Chapman, 1974; Veismanis, 1974; Laier and Schmertmann, 1975; Villet and Mitchell, 1981; Sweeney and Clough, 1990; Huang et al., 1988; Anderson et al., 1991; Hsu and Huang, 1998). Prior to this study commencing there was only one calibration chamber for performing CPTs in unsaturated soils, developed by Miller et al. (2002). However, in conducting a number of CPTs in an unsaturated silt using this chamber, Tan (2005) encountered difficulties due to the design of the specimen formation system.

An important problem concerning calibration chamber testing is the influence of the finite size of the chamber on the CPT results. A number of experimental and theoretical studies have addressed this issue (e.g. Parkin and Lunne, 1982; Bellotti, 1984; Been et al., 1988; Salgado, 1998; Wesley, 2002). However, no reliable methods are available for correction of the CPT results from a calibration chamber so they may be applied to free field conditions.

Cavity expansion theory is one of the most appreciated theoretical approaches for interpretation of the CPT (Yu and Mitchell, 1998). Most contributions have focused on the cavity expansion problem in soils and other materials of infinite radial extent. These include the development of closed form solutions for compressible and incompressible materials (Bishop et al., 1945; Chadwick, 1959; Vesic, 1972; Durban and Baruch, 1976); semi-analytical and the closed form solutions for dilative soils obeying the Mohr-Coulomb yield criterion with a non-associative flow rule (Carter et al., 1986; Bigoni and Laudiero, 1989; Yu and Houlsoy, 1991); a penetration resistance study considering stress rotation around the expanding cavity (Salgado and Prezzi, 2007); numerical solutions for large strain cavity expansion in strain hardening-softening soils (Baligh, 1976; Carter and Yeung, 1985; Yu, 1994; Salgado and Randolph, 2001; Cao et al., 2002) and a semi-analytical solution procedure using the similarity technique for saturated as well as unsaturated soils (Collins et al., 1992; Collins and Stimpson, 1994; Collins and Yu, 1996; Russell and Khalili, 2006b; Russell and Khalili, 2002).

Analyses of the cavity expansion problem in soils of finite radial extent are of particular relevance to the interpretation of CPT results obtained from calibration
chambers. Only a few studies have focused on this problem (Yu, 1992; Yu, 1993; Salgado et al., 1997). However, these studies have a shortcoming in that the constitutive model used to describe soil stress-strain behavior was highly idealized, based on simplistic elastic perfectly plastic assumptions.

1.2 SCOPE AND OBJECTIVES

The main objective of this study is to advance experimental and theoretical bases for interpretation of the CPT in unsaturated soils. Six major contributions are made.

1) A major contribution is the design, manufacture and successful operation of a novel suction controlled calibration chamber suitable for conducting cone penetration tests in dry, saturated or unsaturated soils subjected to stress controlled (or strain controlled) boundary conditions. Unique features of the chamber include the specimen formation system, a modified axial load application system, and the use of an enhanced axis translation technique for application of suction within soil specimens.

2) Conduct and report the results of a targeted program of CPTs to identify salient features relevant to the CPT in unsaturated sands. In particular the effects of suction on cone penetration resistance in unsaturated sands for a range of different initial states is studied. Results for dry, saturated and unsaturated states are compared. In particular the contribution of suction to cone penetration resistance is investigated in relation to the applied net confining stresses.

3) Develop a new analytical procedure for solving the cavity expansion problem in soils of finite radial extent, for the first time incorporating a hardening/softening elastic-plastic constitutive model formulated in the critical state framework. The model is calibrated against results of laboratory tests on saturated and unsaturated sands reported in the literature. The influences of two types of boundary conditions on the pressure required to expand a cavity
as well as the severity of the influences for a range of finite boundary and cavity radii and initial soil states are investigated. Solving the cavity expansion problem in finite media aids the theoretical interpretation of CPT data obtained from calibration chambers.

4) Investigate chamber size and boundary influences on CPT results using the cavity expansion solution procedure and develop a method for correction of CPT results so they resemble what would be measured in free field conditions.

5) Establish a new method for interpreting CPT results in unsaturated sands based on experimental and theoretical investigations. The method will enable an equivalent saturated/dry cone penetration resistance to be estimated at a particular site underlain by unsaturated sand using the measured cone penetration resistance and suction. The equivalent saturated/dry cone penetration resistance enables application of existing CPT correlations to obtain information regarding strength and stiffness. Alternatively, for when both saturated and unsaturated CPT profiles are available for a particular site, the method will enable the indirect measurement of suction’s influence on the effective stress, strength and stiffness.

6) Extend established CPT correlations for sands to account for influences of suction. The new correlations will enable direct characterization of an unsaturated sand from CPT results provided the contribution of suction to effective stress ($\varphi_s$) is known.

1.3 THESIS OUTLINE

Chapter 2 reviews literature relevant to unsaturated soil mechanics, the cone penetration test, methods for analysis of the cone penetration test, calibration chambers, cavity expansion investigations in finite and infinite soil media, interpretation of the cone penetration test in saturated/dry coarse-grained and fine-grained soils and cone penetration tests in unsaturated soils.
Chapter 3 describes the calibration chamber developed as part of this research for conducting laboratory controlled cone penetration tests in unsaturated soils. Equipment and design details as well as control and measurement devices are explained. Detailed design drawings of the calibration chamber are presented in Appendix A.

Chapter 4 presents results of laboratory controlled CPTs conducted in saturated and unsaturated sand. The test material, the specimen formation methods and testing procedures are also described. In particular, the effect of suction on cone penetration resistance is highlighted and discussed.

Chapter 5 details the constitutive model adopted. The model is calibrated for the test sand using the results of triaxial tests reported in the literature. Model simulations are compared to the selected triaxial test results for saturated and unsaturated conditions.

Chapter 6 explains development of a new solution procedure for the cavity expansion problem in soils of finite radial extent. This is followed by generation of results for a sand highlighting the effects of the boundary conditions, extent of the finite boundary and initial state on the cavity limiting pressure.

Chapter 7 presents a novel analytical procedure to quantify calibration chamber size effects. The method builds on the cavity expansion analysis for finite and infinite soil media. The procedure is then used to correct the CPT results (conducted in the calibration chamber) presented in chapter 4 so they resemble those which would be obtained in free field conditions.

Chapter 8 proposes a new method for interpreting the CPT in unsaturated sands. The method is derived based on experimental and theoretical findings of the study. The method may be used to convert cone penetration resistance measured in an unsaturated sand to a corresponding penetration resistance for when the sand is saturated/dry as long as the variation of $F_s$ with depth is known. This enables utilization of existing CPT correlations (developed for saturated/dry conditions) for characterization of unsaturated soils where CPT profiles in saturated/dry conditions are not available. Established CPT correlations for sands are extended to account for influences of
suction. The extended correlations enable direct characterization of an unsaturated sand from CPT results provided the contribution of suction to effective stress ($\chi_s$) is known.

Chapter 9 summarizes the major conclusions drawn from this study and also outlines recommendations for further research.
2.1 INTRODUCTION

The literature review comprises seven main sections. Different aspects of the literature are discussed in these sections on which the contributions presented in chapters 3 to 8 are built on. An outline of the discussions and their correspondence to context of subsequent chapters are noted here.

In section 2.2 general features of unsaturated soil mechanics are reviewed. This includes the concept of suction in unsaturated soils, suction application and measurement techniques, stress state variables, the concept of effective stress in unsaturated soils, constitutive modeling and volume change behavior. The suction application and measurement techniques have been considered in development of the new calibration chamber in chapter 3 and performing the targeted experimental program in chapter 4. The concept of effective stress in unsaturated soils have been used in implementing the constitutive model adopted to cavity expansion analysis in
chapter 6, size effect investigation in chapter 7 and derivation of new interpretation methods for unsaturated soils in chapter 8.

In section 2.3 the general features of the cone penetration test is described. The methods of analyzing cone penetration tests are discussed in section 2.4 and it is explained why the calibration chamber testing and cavity expansion theory have been the most effective approaches in establishing correlations for interpretation of the CPT. A review of the calibration chambers is presented in section 2.5. Certain features of the existing calibration chambers have been utilized in development of the new calibration chamber in chapter 3.

In section 2.6 investigations on the cavity expansion problem in infinite and finite soil media and their shortcomings are reviewed. A novel semi analytical solution procedure for cavity expansion in soils of finite radial extent has been presented in chapter 6 and incorporates a hardening/softening elastic-plastic constitutive model for the first time. Review of the existing solutions and their shortcomings highlights the significance of the theoretical contribution made.

Section 2.7 describes experimental and theoretical studies on calibration chamber size effects. It is highlighted that despite significant efforts in exploring this issue, there are no reliable methods for correction of the CPT data from a calibration chamber so it becomes equivalent to what would be observed in free field conditions. This important need has been addressed in chapter 7 and a new method for correction of calibration chamber size effects is developed. The method is based on spherical cavity expansion analysis in finite soil media.

Section 2.8 presents commonly used methods of interpreting the cone penetration test in saturated or dry soils. Particular emphasis is placed on coarse grained soils. Also, the very few studies of the CPT in unsaturated soils are discussed in this section. The CPT correlations have been extended in chapter 8 to account for suction effects in unsaturated sands. The effective stress concept along with results of experimental and theoretical investigations presented in chapters 4, 6 and 7 are used for derivation of new correlations for the CPT in unsaturated sands.
2.2 FEATURES OF UNSATURATED SOIL MECHANICS

Unsaturated soils are widely encountered in arid and semi-arid regions of the world. These soils consist of three phases: solid particles, air and water occupying the pore space (Figure 2.1). Their stress-strain behavior is complex and is influenced by many factors including externally applied stresses, soil type, structure, density and suction arising from surface tension across the air-water interface within the soil.

In recent years there have been major advances in unsaturated soil mechanics, especially in the development of constitutive models to describe the stress-strain behavior of unsaturated soils incorporating the effects of suction. Research into the stress-strain behaviour, in general, has centered around one of two main approaches: (a) the effective stress approach and (b) the two stress state variables approach. In the effective stress approach, matric suction (the difference between pore air pressure and pore water pressure) and net stress (total stress in excess of pore air pressure) are coupled in an effective stress equation. In the two stress state variables approach the net stress and matric suction are treated as independent variables in explaining the constitutive behavior of unsaturated soils.

Advantages of the effective stress approach over the stress state variables approach are that only a single stress state is required for the complete characterization of the constitutive behavior of an unsaturated soil rather than two independent stresses. The laboratory testing required for determination of soil properties (in terms of effective stress) can be performed in any geotechnical engineering laboratory and is more convenient and quicker than that required to determine soil properties in terms of two independent stress variables (e.g., Loret and Khalili, 2002; Khalili et al., 2004; Russell and Khalili, 2006a). Recently there has been greater interest in using the effective stress approach.

2.2.1 Suction in unsaturated soils

Soil suction is defined as the free energy state of soil water which can be measured in terms of the partial vapour pressure of the soil water (Richards et al., 1956; Aitchison, 1965).
Total suction consists of two components: matric suction and osmotic suction. Matric suction is the difference in pore air pressure and pore water pressure ($u_a - u_w$). Osmotic suction is associated with the salt content and in common geotechnical engineering problems, the changes in osmotic suction are almost negligible compared to changes in matric suction. For example excessive rainfalls may cause instability of unsaturated soil slopes due to reduction in matric suction. A different but also typical problem involving unsaturated soils is the damage to structures caused by the collapse of the foundation soil due to wetting. This phenomenon involves the compression of the soil due to an increase in degree of saturation. An example of foundation collapse occurring in natural soil took place in Via Settem-brini, Naples, on 15 September 2001 (Gens, 2010). A heavy rain storm caused the water to saturate the foundation soil. Maximum settlements of the order of 200 mm were measured from the start of monitoring. In the case where the salt content of the soil is altered, the effect of the osmotic suction may be significant (Fredlund and Rahardjo, 1993).

Therefore, when dealing with unsaturated soils it is often assumed that the change in matric suction is equal to change in total suction.

The relationship between suction and either gravimetric water content ($w$), volumetric water content ($\theta$) or degree of saturation ($S_r$) is described by the soil-water characteristic curves (SWCCs), also referred to as soil-water the retention curves. A number of methods and devices are used for determination of SWCCs including pressure plate, pressure membrane, filter paper, hanging column, chilled mirror hygrometer and centrifuge methods (ASTM D 6836-02).

The general shape of a SWCC is illustrated in Figure 2.2 (after Fredlund et al., 2011). There are two distinct changes in slope along the curve which characterize two essential points in describing the SWCC. The first point is the air-entry value of the soil (for the drying path), where the largest pores start to permit air entry as suction increases. The second point is termed as residual condition and it defines the point where removal of water from soil structure becomes significantly more difficult. Three zones are identifiable, namely the boundary effect zone in the lower suction range, the transition zone between the air-entry value and the residual value, and the residual
zone. Typical SWCCs for different soil types have been illustrated in Figure 2.3 (after Vanapalli et al., 1999).

Laboratory investigation of unsaturated soil behavior requires application, maintenance and measurement of suction in the testing equipment. The axis translation technique was introduced by Hilf (1956) to assist with this and to prevent cavitation in the testing equipment. The procedure involves increasing pore air and pore water pressure such that the pore water pressure will attain a positive pressure value while the soil sample is subjected to a desired matric suction \((u_a - u_w)\). A high air entry value porous disk with air entry value greater than applied matric suction is required to prevent passage of pore air pressure through the measuring system. As pointed out by Bocking and Fredlund (1980) use of the axis translation technique to control soil suction is theoretically sound for soils with totally interconnected pore-air voids. For soils containing significant amounts of occluded pore-air, the actual soil suction will be over-estimated. The other methods of suction control include osmotic and vapour equilibrium techniques. The vapour equilibrium method controls total suction and osmotic technique controls osmotic suction. As this research focuses on unsaturated sands in which pores are well connected, and osmotic suction will be negligible or non-existent, the axis translation is considered to be the most suitable technique.

### 2.2.2 Effective stress in unsaturated soils and associated constitutive models

One of the most significant contributions to soil mechanics has been the introduction of the effective stress principle (Terzaghi, 1936). It is a function of the total stress and pore pressure and converts a multiphase, multi stress state porous medium to a mechanically equivalent, single phase and single-stress state continuum, allowing the application of the principles of continuum solid mechanics to fluid-filled deformable porous media (Khalili et al., 2004).

For unsaturated soils, the effective stress is defined as (Bishop, 1959):

\[
\sigma' = \sigma - u_a + \lambda(u_a - u_w)
\]  

(2.1)
where $\sigma$ and $\sigma'$ represent the total and effective stresses, respectively, $u_a$ is the pore air pressure, $u_w$ is the pore-water pressure and $\chi$ is the effective stress parameter attaining a value of 1 for saturated soils and zero for dry soils. Many attempts have been made in the past to quantify $\chi$ theoretically and experimentally. Bishop (1959) and Bishop and Bligh (1963) assumed a direct correlation between $\chi$ and the degree of saturation, $S_r$. Bishop and Donald (1961) stated that no unique relationship could be found between the degree of saturation and the effective stress parameter. Loret and Khalili (2002) pointed out that $\chi$ should be expressed in terms of the areal fractions of the constituents rather than the volumetric fractions. They indicated that $\chi$ is related to, but not equal to, the degree of saturation and is a function of porosity and the pore-air and pore-water pressures. After analyzing shear strength data for a range of soil types, Khalili and Khabbaz (1998) obtained a unique relationship for the effective stress parameter ($\chi$) in terms of suction ratio ($s/s_e$) for suction ratios less than 12, where $s$ is soil matric suction and $s_e$ represents suction at the transition between saturated and unsaturated states, and may be an air entry or air expulsion value. Russell and Khalili (2006a) extended the relationship to include suction ratios greater than 25.

In the effective stress approach, the shear strength is determined based on the effective stress strength parameters $c'$ and $\phi'$ and a single effective stress defined by Equation (2.1). The advantage of the effective stress approach is that the shear strength of an unsaturated soil is obtained using the same strength parameters as for a saturated soil and this eliminates the need for shear strength tests in an unsaturated state.

Kohgo et al. (1993) showed that the contribution of suction to the effective stress depends on the amount of contact area between the water menisci and the soil grains. This contact area is attributed to the soil structure and is substantially influenced by the matric suction.

Loret and Khalili (2000) presented a comprehensive framework for constitutive modelling of unsaturated soils using the effective stress approach. Qualitative predictions of a model showed the capability of the proposed framework to explain behavioral features of unsaturated soils. Loret and Khalili (2002) developed an elasto-
plastic constitutive model for unsaturated clays within the critical state framework using the effective stress principle and an extension of the Cam clay model. Hardening was assumed to occur due to changes in suction as well as plastic volumetric strains.

Russell and Khalili (2006a) developed a unified constitutive model for unsaturated soils in a critical state framework using the concepts of effective stress and bounding surface plasticity theory. A non associated flow rule of the same general form was used for all soil types. It was also assumed that isotropic hardening/softening occurs due to changes in plastic volumetric strains as well as suction for some types of soils which enabled to capture the phenomenon of volumetric collapse upon wetting. They used the model to simulate to high degree of accuracy the stress-strain behavior of unsaturated sand and kaolin.

Khalili et al. (2008) presented a fully coupled constitutive model for describing cyclic behavior of unsaturated soils including hydraulic and mechanical hysteresis. The elastic plastic behavior due to loading and unloading was captured using the bounding surface plasticity. The coupling between fluid flow and deformation field was established using the effective stress parameters.

Other effective stress based models have been developed by Bolzon et al. (1996), Sheng et al. (2003), Wheeler et al. (2003), Gallipoli et al. (2003a) and Laloui et al. (2003). These models used degree of saturation ($S_r$) as an effective stress parameter. The degree of saturation ($S_r$) is related to $s/s_e$ using a power law. Currently there is no experimental evidence to indicate that $S_r$ to the power of unity can be used as the effective stress parameter.

### 2.2.3 Two stress state variables and associated constitutive models

Fredlund and Morgenstern (1976 and 1977) argued that the mechanical behavior of soil is controlled by the same stress variables which control the equilibrium of the soil structure. They suggested that the effective stress equation for unsaturated soils should be separated into two independent stress state variables. They put forward three possible combinations which can be used as stress state variables for an unsaturated soil: 1) ($\sigma - u_d$) and ($u_a - u_w$), 2) ($\sigma - u_a$) and ($u_d - u_w$), and 3) ($\sigma - u_a$) and ($\sigma - u_w$). Fredlund and Morgenstern (1977) performed a series of triaxial and oedometer null
tests with controlling volume change to investigate suitability of these alternative stress variables. They concluded that the combination of the stress variables \((\sigma - u_a)\) and \((u_a - u_w)\) provide the most satisfactory explanation for experimental observations.

Fredlund and Morgenstern (1977) defined the state of stress in an unsaturated soil in terms of two independent stress tensors: 1) stress tensor described at the macroscopic level, \((\sigma - u_a)\delta_{ij}\), which is averaged over an elementary volume containing all three air, water and solid phases; 2) stress tensor described at the microscopic level, \((u_a - u_w)\delta_{ij}\), which only acts upon soil particles. A separate set of material properties were introduced to each stress tensor.

Alonso et al. (1990) and Wheeler & Sivakumar (1995) developed similar constitutive models for unsaturated clays within the critical state framework using the conventional plasticity theory and the two stress states approach. An associated flow rule was adopted and the yield surface and plastic potentials were expressed in terms of the mean net stress, deviatoric stress and matric suction. Isotropic elasticity rules were adopted, similar to those used in the Cam clay type models, but the net stress replaced the effective stress and an additional volumetric strain component was expressed in terms of matric suction.

Cui and Delage (1996) presented a model for compacted silt and captured the influences of anisotropy by using an inclined ellipse as a yield function along with a non-associated flow rule. Rampino et al. (2000) adopted the model of Jefferies (1993) applying some of model features of Alonso et al. (1990) to capture the observed stress-strain behavior of an unsaturated silty sand.

The two stress state models can capture many behavioral features of unsaturated soils such as the dependence of the size of yield surface on suction and volumetric collapse upon wetting. However, as pointed out by Loret and Khalili (2002) they fail to reproduce an important characteristic feature of unsaturated soils: plastic volumetric change followed by an elastic response observed in normally consolidated soils subject to increasing values of suction (Fleureau et al., 1993). Another major aspect which cannot be incorporated in two stress state models is the role of air entry value on the behavior of unsaturated soils. In the effective stress based constitutive models the air
entry or air expulsion value plays a key role in the definition of the effective stress and hence the simulation of unsaturated soil behaviour, as discussed in section 2.2.2.

### 2.2.4 Volume change behavior of unsaturated soils

Drying (increase in suction) causes shrinkage in unsaturated soils and wetting (decrease in suction) may result in volumetric swelling or collapse of the soil structure depending on the confining pressure applied on the soil (e.g., Fleureau, 1993; Jennings and Burland, 1962; Matyas and Radhakrishna, 1968). The observed response of collapse upon wetting in unsaturated soils has been used in the literature to question validity of the effective stress concept in unsaturated soils (eg., Fredlund and Morgenstern, 1977; Alonso et al., 1990; Gens et al., 1995).

However, as pointed out by Loret and Khalili (2000), this phenomenon is due to the shift in preconsolidation pressure of unsaturated soils with suction. Indeed suction affects the effective stress as well as yield stress of the unsaturated soil. A linearly equivalent elastic framework cannot be used to explain irrecoverable volumetric deformations such as collapse and an appropriate plastic model should be invoked.

### 2.3 GENERAL DESCRIPTION OF THE CONE PENETRATION TEST

The cone penetration test (CPT) involves pushing a cone penetrometer into the soil and recording continuous measurements of cone resistance and sleeve friction with depth. The first inception of a cone penetrometer was in 1932 in the Netherlands. Initial cone systems were of mechanical type and were pushed by hand (Broms and Flodin, 1988). Over the past decades there have been significant advances in the equipment. Electrical cone penetrometers provide more precise, more reliable, simpler and faster testing procedures (Mayne, 2007). An advanced CPT system includes the following components: an electrical penetrometer, hydraulic pushing system with rods, cable or transmission device, depth recorder and data acquisition unit.

The standard test equipment (ASTM D 3441) consists of a cone with an apex angle of 60º, tip area of 10 cm² and a friction sleeve area of 150 cm² located above the cone.
The cone is pushed into the ground using the push rods maintaining a rate of 20 mm/s. Miniature cone penetrometers are also widely used for both research and consulting purposes.

The cone (tip) resistance \( (q_c) \) is defined as the measured total force acting on the cone \( (Q_c) \) divided by the total projected area of the cone \( (A_c) \). Sleeve friction \( (f_s) \) is defined as total force acting on the sleeve \( (F_s) \) divided by the surface area of the sleeve. The friction ratio \( (R_f) \) is expressed as the ratio (in percentage) of the sleeve friction \( (f_s) \) to cone resistance \( (q_c) \) measured at the same depth.

In the piezocone penetrometers, pore water pressure \( (u_w) \) may be measured at different locations on the equipment and these pore pressures are known as: \( u_1 \) (on the cone), \( u_2 \) (behind the cone) and \( u_3 \) (behind the friction sleeve). A schematic diagram of a typical piezocone with pore water pressure measurement on the cone is shown in Figure 2.4.

Due to inner geometry of a cone penetrometer the ambient pore water pressure acts on the shoulder area behind the cone and on the ends of the friction sleeve. This is illustrated in Figure 2.5 (Lunne et al., 1997) and referred to as the unequal area effect. The total resistance measured by cone is influenced by this phenomenon. The unequal area is represented by the cone area ratio \( (a) \), which is approximately equal to the ratio of the cross-sectional area of the load cell or shaft \( (A_n) \) to projected area of the cone \( (A_c) \). The corrected total cone resistance \( (q_T) \) is expressed as (Robertson and Campanella, 1983b):

\[
q_T = q_c + u_2(1 - a)
\]  

(2.2)

where \( u_2 \) is the pore pressure measured behind the cone.

For saturated sands, drained conditions prevail during cone penetration testing meaning no excess pore pressures are generated at the cone tip during penetration. For this reason it is customary to interpret CPT data using \( q_c \) instead of \( q_T \) (Lunne et al., 1997), even in terms of effective stresses present in the sand. However, the unequal
end-area correction can be significant in soft fine-grained soil where $q_c$ is low relative to the high water pressure around the cone due to the undrained penetration process.

Although pore-pressure measurements are common with CPT (sometimes referred to as CPTu) the accuracy and precision of the cone pore pressure measurements for onshore testing are not always reliable due to problems associated with saturation of the porous element (Robertson, 2009). The porous elements are usually saturated with silicon oil or glycerin but for a CPTu performed through unsaturated soil layers, suction can be sufficient to de-saturate the pore pressure sensor. The use of aforementioned viscous liquids can alleviate the loss of saturation but do not remove the problem entirely.

The CPT has various applications in saturated soils including sub-surface stratigraphy identification, soil classification, determination of shear strength and state characteristics, direct applications in design of shallow and deep foundations and liquefaction assessment (Lunne et al., 1997; Mayne, 2007 and Robertson, 2009). Also in recent years by introducing a number of additional sensors and devices to the CPT, there has been a notable increase in geo-environmental applications. The common interpretation methods for the CPT are discussed in section 2.8.

2.4 METHODS FOR ANALYSIS OF CONE PENETRATION

Different approaches for analyzing cone penetration have been reported in the literature including bearing capacity theory, cavity expansion theory, steady state deformation, incremental finite element analysis and calibration chamber testing. In this section, general features of the analysis methods as well as advantages and disadvantages are discussed. The common CPT correlations derived from these methods are discussed in section 2.8.

2.4.1 Bearing capacity theory

One of the first approaches for the analysis of the cone penetration test has been bearing capacity theory. The cone resistance is assumed to be equal to the collapse
load of a deep circular foundation in soil. Two analytical approaches used to determine the cone resistance are limit equilibrium and slip line analysis.

In the limit equilibrium method the failure load is determined firstly by defining a failure mechanism around the tip of the penetrometer and then by analyzing global equilibrium of the soil mass. In this method the mechanism of failure is defined by assuming rigid plastic models. Based on plane strain solutions a number of correlations have been developed for determination of cone factors $N_c$ and $N_q$ in cohesive and cohesionless soils respectively (e.g., Meyerhof, 1961; Janbu and Senneset, 1974; Durgunoglu and Mitchell, 1975). Although the limit equilibrium analyses are simple, the obtained solutions are only approximate as they ignore the effect of soil stress-strain behaviour and elastic deformations.

In the slip-line method, a yield criterion is combined with the equations of equilibrium to form a system of differential equations of plastic equilibrium in the soil mass (e.g., Sokolovskii, 1965; Houlsby and Wroth, 1982; Koumoto and Kaku, 1982; Koumoto, 1988). From the basic slip-line differential equilibrium equations, a slip-line network can be developed and collapse load may be determined. Slip-line analysis is more rigorous than the limit equilibrium method as it satisfies both the equilibrium equations and the yield criterion everywhere within the slip-line network. The limit equilibrium method only satisfies the global equilibrium. A major drawback of the slip-line method is that the stress distribution outside the slip-line surface could not be defined.

Another limitation of the bearing capacity approaches is that the influence of the cone penetration process on the initial stress states around the shaft is ignored in these methods. In particular, the horizontal stress tends to increase around the cone shaft after cone penetration and the influence of this change on the cone resistance is not accounted for in both limit equilibrium and slip-line methods.

2.4.2 Steady state approach

The steady state approach involves estimating a strain field around a cone penetrometer. In the solutions, penetration was considered as a steady state flow of soil
past a fixed cone penetrometer (e.g., Baligh, 1985; Houlsby et al., 1985; Tumay et al., 1985; Teh, 1987; Sagaseta and Houlsby, 1988). This means that the stress and strain fields in the soil are not changed with time from the point of view of the cone tip if homogeneous soil conditions are present. Although most solutions obtained using the steady state approach is based on perfectly plastic soil models, it is also possible to include strain hardening critical state models in the calculation. Since the initial estimation of a flow field for frictional soils is very difficult, the application of this method has been restricted to cone penetration in undrained saturated clays.

2.4.3 Incremental finite element method

Significant contributions have been made to analyze the cone penetration tests in soils using incremental displacement finite element methods. In small strain incremental finite element approaches, in situ stress conditions were assumed and an incremental plastic collapse calculation was implemented. The collapse load was assumed to be equal to penetration resistance (e.g., de Borst and Vermeer, 1982; Griffiths, 1982). This approach is considered to underestimate cone penetration resistance, as during the cone penetration, high lateral stresses tend to develop next to the shaft of the cone and buildup of the stresses surrounding the shaft could not be predicted by a small strain analysis.

More realistic large strain finite element analyses were performed by Kiousis et al. (1988) and Budhu and Wu (1992) for clays and Cividini and Gioda (1988) for sands. As pointed out by van den Berg (1994), in using these analyses it is required to determine new locations of boundary nodes which makes the analysis very complicated and robustness of the numerical procedure is not clear.

Van den Berg (1994) presented a large strain analysis of the cone penetration test in both clay and sand using Eulerian formulation. In the Lagrangian-Eulerian formulation, the finite element mesh is fixed in space, while the material streams through it. The Drucker-Prager and von Mises failure criteria were used to simulate the penetrometer passing from sand into soft clay and vice versa. A minor drawback of this approach is that some numerical diffusion was introduced by the smoothing technique inherent to the convection algorithm.
Yu et al. (2000) presented a finite element procedure based on a steady state approach for undrained penetration in clay. The steady state finite element analysis focuses on the total displacements experienced by soil particles at a particular instant in time during the cone penetration test. In their method, the penetrometer was placed in a pre-bored hole and only a few steps of penetration were modeled. A major shortcoming of this approach is that transient deformation of the soil body around the cone is neglected.

Further large strain finite element analyses have been presented by Susila and Hryciw (2003), Huang et al. (2004), Liyanapathirana (2009), Walker and Yu (2010). Contact elements were used at the soil-pile interface to allow penetration to large depths to be achieved. A limitation of the aforementioned studies is that a systematic calibration between the predicted $q_c$ resistance and field measurements was not conducted. In addition a large strain incremental finite element analysis of the CPT is very time consuming and therefore requires considerable computational resources.

However as pointed out by Yu and Mitchell (1998), due to the significant errors and numerical difficulties associated with collapse load calculations, incremental finite element methods could not provide a comprehensive solution for cone penetration in soils. Cavity expansion theory therefore remains one of the most common approaches to theoretical investigation of the CPT. Calibration chamber testings also play an important role in establishing correlations between CPT results and soil properties. Calibration chambers and cavity expansion theory are reviewed in the following sections.

2.5 CALIBRATION CHAMBERS

Analysis of data obtained in the field is not suitable on its own for developing correlations for interpreting CPT results as many uncertainties surround the heterogeneity of soil, stress history, boundary conditions and stress state of soil. Calibration chambers have been used in the past to develop correlations under laboratory controlled conditions but mainly for saturated or dry soils (Ghionna and Jamiolkowski, 1991). Advantages of calibration chamber testing over field testing
include repeatability of specimen properties and tests results, controlled and known boundary conditions and stress history.

The first large flexible wall calibration chamber was designed by Holden (1971) at the Country Roads Board (CRB) Australia, which consisted of a double wall cylinder and a membrane enclosing a sand specimen. This allowed the system to impose a zero average lateral strain boundary condition on the specimen during testing, or alternatively a constant vertical and/or horizontal stress boundary condition. Vertical stress was applied to the specimen by a pressurised cushion acting below a base plate of the same diameter as the specimen. The CRB chamber could accommodate a cylindrical specimen of 760mm in diameter and 910mm in height. Unavoidable boundary effects in the CRB chamber when testing the 20cm$^2$ penetrometer led to design of a much larger diameter calibration chambers at the University of Florida, USA, (Laier et al., 1975) (accommodating a specimen of 1220mm in diameter and 1220mm in height) and Monash University, Australia (Chapman, 1974)(1200mm in diameter and 1800mm in height). The larger specimen heights enabled plateaus to be obtained in the sleeve friction resistances when testing the 10cm$^2$ Fugro penetrometer. In the Monash University chamber, automatic recording of all pressures as well as the vertical movement of the piston using a displacement transducer was possible.

Similar calibration chambers were manufactured in different sizes for testing saturated or dry sand specimens (e.g., Veismanis, 1974; Parkin and Lunne, 1982; Sweeney and Clough, 1990). The pluvial deposition method (Jacobsen, 1976) was used for preparation of sand specimen in these studies. This method seems to be the most widely accepted method for sand specimen preparation because of the repeatable homogenous specimens formed and the relative simplicity of the equipment. Generally the pluviation system consists of a hopper and diffuser system. Density of the prepared specimens is controlled by sand flow rate and drop height (Rad and Tumay, 1987; Lo Peresti et al., 1992).

A calibration chamber with an advanced system of specimen saturation and control system was designed and built at ENEL C.R.I.S, Italy (Bellotti, et al., 1982). The sand specimen was enclosed by rubber membranes at side and base. The side membrane
was sealed around an aluminium plate which formed the top boundary of the specimen and transferred the trust of the chamber piston from the sand to the lid. The specimens were de-aired and then saturated using CO₂. In this technique a slow rate flow of CO₂ is used to fill the voids in place of the air and then de-aired water at atmospheric pressure substitutes CO₂ which is more soluble in water than air. No tensional history is applied to the specimens in this method (compared to vacuum application techniques) but long duration of the saturation procedure is a significant disadvantage in preparation of large specimens. Vertical stress was applied to the specimen by pressurized water via chamber piston and the lateral stress was applied by a water-filled annular space surrounding the specimen.

A significantly different calibration chamber for testing sand specimens was designed by Villet and Mitchell (1981). It was capable of imposing only a constant stress boundary condition on the sand specimens. The vertical pressure was applied via a pneumatically controlled bladder on top of the chamber instead of the pistons used at bottom of the CRB and other chambers.

Hsu and Huang (1998) developed an axisymmetric field simulator to perform CPTs with minimal boundary effects in dry sand. The system consisted of a stack of rings to house the sand specimen lined within an inflatable silicone rubber membrane. The boundary expansion was measured and stress was adjusted pneumatically at each ring level individually during the cone penetration.

The application of calibration chambers to cohesive soils has been limited to a few studies. This is essentially due to the time consuming and laborious process involved in the preparation and consolidation of large cohesive soil specimens. The complexities involved in the instrumentation for monitoring of pore pressures and maintaining saturation add to the difficulty of performing tests in cohesive soil specimens.

The first calibration chamber for testing cohesive soils was developed by Huang et al. (1988). It included instrumentation and porous disks at the top and bottom of the specimens enabling double drainage and pore pressure monitoring during testing. The
annular space between the specimen and the chamber inner wall as well as that between the inner and outer walls was filled with de-aired water. Both ends of the specimen were isolated from the cell water, and the stresses in the vertical and horizontal directions could be controlled independently. The clay specimens were prepared using slurry and chamber consolidation systems. This chamber could accommodate only small specimens of 200mm in diameter and 337mm in height and therefore boundary effects were unavoidable.

A larger calibration chamber for cohesive soils capable of simulating constant stress boundary conditions was designed by Anderson et al. (1991). Clay bed preparation procedure included initial consolidation of slurry under $K_0$ conditions and then further consolidation using equal or unequal horizontal and vertical stresses. The consolidometer could accommodate clay specimens of 785mm in diameter and 1700mm in height. The main limitation of this chamber was capability of only imposing constant stress boundary condition.

Tumay and de Lima (1992) developed a large double walled calibration chamber for testing cohesive soils at the Louisiana State University, USA. It could accommodate specimens of 525mm in diameter an 815mm high and was capable of simulating constant stress and zero strain boundary conditions. This chamber was initially designed to test compacted specimens and later two slurry consolidometers were developed to prepare cohesive soil specimens by the slurry consolidation technique (Kurup, et al., 1994).

A calibration chamber for performing CPTs in unsaturated soils was designed and built at the University of Oklahoma (Miller et al., 2002). Radial and axial stresses may be applied and controlled independently: the radial stress by air pressure pushing on the specimen membrane and axial stress by a hydraulic ram pushing against the specimen base. Also, high air entry value porous disks used in the configuration of this chamber enabled application and maintenance of suction in the specimen using the axis translation method.
However, in conducting a number of CPTs in unsaturated silt using the Oklahoma chamber, Tan (2005) encountered difficulties. Inducing suction in a specimen using the axis translation method was found to be very time consuming so instead specimens were prepared with moisture contents related to target suction. Also, the specimen former that encloses specimens during fabrication was not rigid enough which led to specimens expanding radially during compaction. As a consequence, high specimen densities could not be reached.

2.6 CAVITY EXPANSION INVESTIGATIONS
Solutions to the problem of cavity expansion in soils have received a great deal of attention in geotechnical engineering. Perhaps the most common applications concern the interpretation of in situ test results such as the CPT (Yu, 2000). Bishop et al. (1945) pointed out that the pressure at the wall of an expanding cavity in a soil mass approaches a limiting value at large strains and this is analogous to cone resistance ($q_c$) of the cone penetration test.

2.6.1 Cavity expansion in infinite soil media
There have been many notable contributions to the cavity expansion problem in soils and other materials of infinite radial extent. These include the development of closed form solutions for compressible and incompressible materials (e.g., Bishop et al. 1945; Chadwick, 1959; Durban and Baruch, 1976). In these studies, the constitutive models used for the stress-strain behavior were quite simple such that closed form solutions could be developed.

Vesic (1972) suggested approximate solutions for cavity expansion in compressible soils. The behaviour of the plastic zone surrounding the cavity was defined by the Mohr-Coulomb yield criterion. Volume change was initially introduced to the analysis as a known value and then determined for stress change through an iterative procedure. Baligh (1976) extended the work by using a curved Mohr-Coulomb failure envelope and concluded that ignoring the curvature of the envelope may result in significant error in the cavity limit pressure.
Carter et al. (1986) developed a closed form solution for the limit pressure at the cavity wall using Mohr-Coulomb model. A non-associative flow rule and constant friction and dilation angles were assumed. In expressing time derivations, the convected component was ignored which enabled development of closed form solutions. However, Collins and Wang Yan (1990) pointed out that failure to include the convected component can result in errors up to 15%. Yu and Carter (2002) showed that the errors introduced due to neglecting convected part may increase when friction and dilation angles become very large. Bigoni and Laudiero (1989) obtained a closed form solution by adopting a constitutive model the same as that of Carter et al. (1986) and neglecting elastic deformations in the plastic region.

Yu and Houlsby (1991) used the logarithmic strain definition in the cavity expansion analysis which enabled taking into account the large strain effects. The Mohr-Coulomb constitutive model was adopted along with a non associative flow rule. The cavity pressure-strain relationship was obtained using a series expansion.

Numerical solutions for large strain cavity expansion in strain hardening-softening soils were also presented (e.g., Carter and Yeung, 1985; Yu, 1994; Salgado and Randolph, 2001; Cao et al., 2002). Cudmani and Osinov (2001) obtained numerical solutions for large strain expansion of a cavity in infinite granular soils using the hypoplastic constitutive relation.

Collins et al. (1992) and Collins and Stimpson (1994) developed a sound semi analytical solution procedure for cavities expanded from zero initial radius in infinite soils. The procedure, referred to as similarity technique, is based on the fact that such problems have no characteristic length so stresses and strains surrounding the expanding cavity remain self similar. In this approach the system of governing differential equations is solved simultaneously using a differential equation solver. Based on kinematics of cavity expansions Collins and Yu (1996) used the similarity technique to solve undrained expansions from non-zero initial radii.

Russell and Khalili (2002) used similarity technique of Collins and coworkers to investigate the effect of particle crushing on cavity expansion in sands. Specifically the
sand state was defined in terms of a critical state line capturing features of sand behavior for stresses that are lower and higher than those required for particle crushing. For initial states typical of quartz sands it was shown that particle crushing occurs and cause a significant reduction in the stress surrounding the cavity.

Russell and Khalili (2006b) extended the application of similarity technique to unsaturated soils using a unified bounding surface plasticity constitutive model and concepts of effective stress. The cavity expansion results were generated for constant suction and constant moisture content conditions. It was shown that suction significantly increases the pressure required to expand a cavity from that required for saturated conditions.

2.6.2 Cavity expansion in finite soil media
There have been far fewer contributions which have focused on the cavity expansion problem in soils of finite radial extent.

A solution of the cavity expansion problem in finite soils and its relevance to the analysis of pressuremeter tests were presented by Jewell et al. (1980) and Fahey (1986), however the elastic deformation in plastic zone is ignored in these analyses and therefore these solutions are only approximate small strain solutions.

Yu (1992) and Yu (1993) presented analytical solutions for expansions in soil media having the shape of a thick walled cylinder and a hollow sphere subjected to a constant stress boundary condition. In these studies, it was only possible to use the simplistic elastic perfectly plastic Mohr-Coulomb soil model in the solution procedure, in which friction angle and dilation angle are constant.

Salgado et al. (1997) considered expansion of cylindrical cavities in finite and infinite media again using a Mohr-Coulomb type constitutive model and incorporated stress rotation around the expanding cavity. An operative flow number was introduced which accounted for variation of friction angle and dilation angle. Salgado et al. (1998) supposed that cavity pressure is analogous to cone penetration resistance and used the cylindrical cavity expansion results to study how cone penetration resistance measured
in a calibration chamber is influenced by the size of the chamber. A constant radial stress boundary condition was assumed. However, no results have been obtained for the zero displacement boundary condition or for spherical cavities.

2.7 CALIBRATION CHAMBER SIZE EFFECTS

An important issue concerning cone penetration testing in a calibration chamber is that the size of the test specimen is limited and that penetration resistance ($q_c$) may be influenced by the boundary conditions imposed by the chamber. Hence, a measured $q_c$ may not represent that for the field condition even when the test soil, the initial properties, and the stress states are identical.

A significant contribution to this issue has been the experimental studies of Parkin and Lunne (1982). They conducted cone penetration tests in two different calibration chambers using penetrometers of two different sizes to investigate the influence of size and boundary conditions on the measured cone resistance. For the equipment used, diameter ratios ranged from 21.4 up to 48.3. Figure 2.6 presents the observed size effects in this study. For normally consolidated dense specimens, the influence of chamber size is minimal for diameter ratios of larger than 50 when the results for different boundary conditions converge. For the overconsolidated specimens a plateau was not reached although the results were independent of the boundary condition. For the loose sands chamber size effects is not apparent.

Further considerations through experimental observation have been given to the calibration chamber size effects by Bellotti (1984), Parkin (1988), Been et al. (1988) and Ghionna and Jamiolkowski (1991). The general conclusion drawn from these studies is that for the boundary effects to become negligible, the ratio of specimen diameter to cone diameter ($R_D$) needs to be in excess of about 35 for loose sands and in excess of 60 for dense sands. Lesser values of $R_D$ may result in significant reductions in the measured values of the cone resistance.

Schnaid and Houlsby (1991) investigated the chamber size effects using three cone pressuremeters of cross-sectional areas 15cm$^2$, 10cm$^2$ and 5cm$^2$ (corresponding to
chamber to probe diameter ratios of 38, 27 and 22 respectively). Sand specimens of three different relative densities were utilized. For the dense and medium sand, an increase in the normalized limit pressure with increasing of the diameter ratio was observed. For the loose sand there was no significant influence of the chamber size.

Explanations of the problem have focused mainly on the constraining effect of the chamber boundaries as the cavity is created in the soil by the penetrating cone. Salgado et al. (1998) used cavity expansion analyses to explain the size effects in calibration chambers, although their analyses were limited to cylindrical cavity expansions. Considering the experimental studies in the literature it can be concluded that the cylindrical cavity expansion analysis overestimates the size effects particularly for loose sands.

An alternate explanation was put forward by Wesley (2002), who argued that the change in the vertical stresses arising from the downward force of the cone penetrometer during penetration may cause a reduction of cone resistance with decreasing chamber size.

Mayne & Kulhawy (1991) presented empirical correction factors that consider $R_D$ and soil relative density as:

$$q_c/q_{c,\infty} = [(R_D - 1)/70]^{D_c/200}$$

(2.3)

Jamiolkowski et al. (2003) suggested an empirical correlation for correction of size effects as:

$$q_c/q_{c,\infty} = \left(aD_r^b\right)^{-1}$$

(2.4)

where $a$ and $b$ are coefficients dependant on $R_D$. However, the significant influence of confining stress has so far been ignored.
Despite these efforts, there are no reliable methods for correction of the CPT data from a calibration chamber so it becomes equivalent to what would be observed in free field conditions or an infinitely large chamber.

2.8 INTERPRETATION OF CONE PENETRATION TESTS
This section presents commonly used CPT interpretation correlations. The interpretive procedures and correlations linking CPT results to soil properties for saturated or dry soils are placed into two categories here: interpretation in coarse grained soils and interpretation in fine grained soils. No correlations exist for interpretation of CPT data in unsaturated soils, although the very few studies which have started to address this issue are discussed.

2.8.1 Interpretation of CPT in saturated/dry coarse-grained soils
Cone penetration in saturated coarse grained soils, such as sands, occurs under drained conditions and no excess pore water pressure is generated during the penetration process.

2.8.1.1 Relative density
Relative density ($D_r$) is commonly used by practitioners to describe sand state. Numerous calibration chamber studies have been conducted to establish correlations between cone penetration results, in situ stress state and relative density of sands (e.g., Schmertmann, 1976; Chapman and Donald, 1981; Villet and Mitchell, 1981; Baldi et al., 1982).

Based on an extensive calibration chamber testing program a correlation for determination of relative density of sands was proposed by Baldi et al. (1986):

$$D_r = \frac{1}{C_2} \ln \left( \frac{q_c}{C_0 (\sigma')^{C_3}} \right)$$  \hspace{1cm} (2.5)
where $C_0$, $C_1$ and $C_2$ are constants unique to the soil tested. The values of $q_c$ and $\sigma'$ are expressed in kPa. They suggested $C_0 = 157$, $C_1 = 0.55$ and $C_2 = 2.41$ be used along with initial vertical stress for normally consolidated Ticino sand. Also $C_0 = 181$, $C_1 = 0.55$ and $C_2 = 2.61$ were found to suit over-consolidated Ticino sand if used with initial mean effective stress. Similar correlations in terms of initial effective stress have been proposed by others (e.g., Villet and Mitchell, 1981; Jamiolkowski et al., 1985; Kulhawy and Mayne, 1990; Jamiolkowski et al., 2001).

As pointed out by Baldi et al. (1986) and Jamiolkowski et al. (2001), a correlation using vertical effective stress is only valid for normally consolidated sands and correlations based on mean effective stress apply more widely to both normally and over-consolidated sands. Only Houlsby and Hitchman (1988) have used a power law in terms of horizontal effective stress.

In a review of calibration chamber tests Robertson and Campanella (1983a) showed that the correlations between cone resistance, effective stress and relative density were all similar in nature but were strongly influenced by sand compressibility (Figure 2.7).

Therefore different power laws may apply to sands with various compressibility values as is evident through the range of correlations proposed in the literature. For example the correlations proposed by Schmertmann (1976) represent the results of tests performed on Hilton Mines sand, which is a highly compressible quartz, feldspar, mica mixture with angular grains. The curves suggested by Villet and Mitchell (1981) represent results of tests conducted on Monterey sand with relatively low compressibility. Ticino sand used by Baldi et al. (1981, 1986) was a quartz, feldspar, mica mixture with subangular particles and appears to have a moderate compressibility. Generally sands with angular grains tend to be more compressible than sands having rounded grains.

Sands with a high compressibility tend to have a lower cone resistance than sands with a low compressibility and the same relative density. However, cone resistance can be uniquely related to relative density for any given sand.
2.8.1.2 State parameter

The state parameter is defined as the difference in the void ratio of a soil at a given mean effective stress and the void ratio on the critical state line at the same mean effective stress. Void ratio and stress level are the most important physical conditions which define the current state of the soil and therefore control its behavior. The state parameter combines the influence of void ratio and stress level for sands with reference to critical state. The parameter aids characterization of sand behavior and some key behavioral properties of sand can be characterized by the state parameter irrespective of the median grain size and mineralogy (Been & Jefferies, 1985).

Based on calibration chamber studies, Been et al. (1986, 1987) proposed a procedure for estimating the state parameter from cone penetration tests in sand. They reviewed test data for different sands and showed that a common relationship between normalized cone resistance and state parameter exists (Figure 2.8). The state parameter can be calculated as:

\[
\frac{q_c - \sigma_{mean}}{\sigma_{mean}^f} = k \cdot \exp(-m\zeta)
\]  

(2.6)

Where \( \zeta \) is state parameter and \( m \) is the slope of the normalized \( q_c - \zeta \) relationship and \( k \) is the normalized \( q_c \) value at \( \zeta = 0 \).

The interpretation procedure includes determining the critical state line through a series of triaxial tests. For primary interpretation it may be possible to use results from tests on previously tested sands if it can be assumed to behave similarly to the one being considered.

As discussed in 2.8.1 a major drawback of relative density correlations is that they are strongly influenced by sand compressibility. In the critical state approach the influence of compressibility on the correlations can be accounted for through the slope of the critical state line, however the use of linear normalization procedure can be questioned and may need further validation. The other disadvantage is requirement of performing companion triaxial tests for determining the critical state line. As pointed out by
Jefferies and Been (2006) the problem of evaluating the sand state from CPT response is complex and depends on several soil parameters including shear stiffness, shear strength, compressibility and plastic hardening. They described how the state can be evaluated using a combination of laboratory and in situ tests.

2.8.1.3 Shear strength characteristics
Based on calibration chamber test results, Robertson and Campanella (1983a) showed that the peak friction angle for clean, uncemented silica sands could be estimated from the normalized cone resistance (Figure 2.9). In their study the peak friction angle values were obtained from drained triaxial compression tests performed at confining stresses approximately equal to the horizontal stresses in the calibration chamber before cone penetration. The correlations of Robertson and Campanella (1983a) provide reasonable estimates of friction angle for normally consolidated, moderately compressible, predominantly quartz sands. For highly compressible sands, the chart would tend to predict conservatively low friction angles. Kulhawy and Mayne (1990) modified the relationship considering a larger data base of calibration chamber tests conducted on different sand types. Other relationships for peak friction angle were proposed based on bearing capacity theory (e.g., Janbu and Senneset, 1974; Durgunoglu and Mitchell, 1975).

2.8.2 Interpretation of CPT in saturated fine-grained soils
Cone penetration in saturated fine grained soils, such as clays, is essentially undrained and the correlated characteristics are based on soil undrained behavior. For the CPTs performed under undrained conditions pore water pressure is generated and it is important to measure pore pressure and correct the CPT results using Equation 2.2. The pore pressure measurements can also be used for determination of in situ state which is discussed in section 2.8.3.2.

2.8.2.1 Undrained shear strength
The general correlation for evaluating undrained shear strength ($s_u$) from total cone resistance is expressed as:
\[ s_u = \frac{(q_r - \sigma_{w0})}{N_k} \]  

(2.7)

where \( N_k \) is a dimensionless cone factor.

Several experimental investigations have been conducted to obtain the value of the cone factor. Kjekstad et al. (1978) suggested that for non-fissured over-consolidated clays \( N_k \) may be assumed as 17. They obtained the reference undrained shear strength \( (S_u) \) from triaxial compression tests. Lunne and Kleven (1981) showed that for normally consolidated marine clays the cone factor varies between 11 and 19. In their study the reference undrained shear strength was obtained from field vane tests. Robertson and Campanella (1983) suggested using \( N_k \) values of 15 for preliminary assessment of undrained shear strength and for sensitive clays, the \( N_k \) value should be reduced to around 10 or less depending on the degree of sensitivity. It is more difficult to establish similar correlations in stiff over-consolidated clays because of the effects of fabric and fissures on the response of the clay. Aas et al. (1986) presented correlations between cone factor and plasticity index. It was observed that cone factor increased as the plasticity increased and the cone factor varied between 11 and 18 for the range of plasticity indices considered.

Senneset et al. (1982) put forward a different approach for estimation of undrained shear strength using effective cone resistance. The effective cone resistance is defined as the difference between the measured cone resistance and pore water pressure measured immediately behind the cone \( (u_2) \). They showed that effective cone factor varied between 6 and 13. In soft normally consolidated clays, the total pore pressure generated behind the cone is usually about 90% of the measured cone resistance and a major disadvantage of using effective cone resistance to interpret undrained shear strength in these soils is that the effective cone resistance is a very small quantity, extremely sensitive to small errors in cone resistance or pore water pressure measurements.

Significant theoretical contributions based on cavity expansion theory and bearing capacity theory have been made to determine the cone factor (e.g., Meyerhof, 1951; Vesic, 1972; Baligh, 1975; Teh, 1987; Konrad and Law, 1987), resulting in cone...
factors similar to those observed experimentally. The solutions based on bearing capacity theory incorporate the incipient failure of a rigid plastic material and are highly dependent on the assumed shape of the plastic zone.

2.8.2.2 In situ state

For fine grained soils, the in situ state is expressed in terms of over consolidation ratio (OCR). The OCR is usually defined as the ratio of the maximum past effective consolidation stress to the present effective overburden stress. This definition is only valid for mechanically overconsolidated soils where the overconsolidation was created by removal of the overburden stress. For cemented and aged soils the OCR may more appropriately defined as the ratio of the yield stress and the present effective overburden stress (Robertson, 2009).

For piezocone tests where pore pressures are measured both on the cone tip \(u_1\) and behind the cone \(u_2\), Sully et al. (1988) showed that the normalized pore pressure difference could be related to OCR. They observed that the pore pressure measured on the tip or face of the cone is always higher than that measured behind the cone and as the over-consolidation ratio increases, the difference between the normalized pore pressure measured on the face and at the base of the cone increases. Based on this finding a linear relationship between OCR and the pore pressure parameter (PPD) was suggested as:

\[
OCR = 0.66 + 1.43(PPD)
\]  

(2.8a)

where PPD is the normalized pore pressure difference and defined as:

\[
PPD = \frac{u_1-u_2}{u_0}
\]  

(2.8b)

However, the pore-pressure parameter, PPD, was shown to give good relationship for a limited number of soils with OCR<10. For the highly over-consolidated soils, the PPD-OCR relationship does not agree with field data. This may be due, in part, to
partial drainage of excess pore pressure during penetration caused by the soil microstructure.

Mayne (1991) conducted a review of methods for interpretation of OCR from the CPT and proposed a correlation for determining OCR based on cavity expansion theory. The proposed correlation is applicable to piezocone data with pore pressure measurement behind the cone ($u_2$) and is of the form:

$$OCR = 2 \left[ \frac{1}{(1.95M+1)} \left( \frac{q_t-u_2}{\sigma_{v0}} \right) \right]^{1.33}$$  \hspace{1cm} (2.9)

where $M$ is the slope of the critical state line.

Mayne (1991) presented a good agreement between predicted and measured profiles of OCR for different sites underlain by variety of clay types.

2.8.3 CPT in unsaturated soils

Hryciw and Dowding (1987) conducted an experimental investigation in Ottawa (quartz) sand although it is important to note that a much smaller and less sophisticated chamber was used than that used in present study. Specimens were prepared to a relative density of $D_r = 50\%$ and a range of degrees of saturation ($S_r$). The specimens in the chamber were not subjected to external confining stresses during the tests, rather, a zero displacement boundary condition was used. Cone penetration resistance at 0.3 m depth was measured (where $p_{r0} \approx 5$kPa corresponding to a unit weight of about 16kN/m$^3$). The results of Hryciw and Dowding (1987) indicate that for $S_r$ values larger than about 0.65, suction induced in the specimens had negligible effect on the cone resistance. However, for $S_r$ less than about 0.1 there was a significant increase in cone penetration resistance.

Lehane et al. (2004) presented a series of CPT results for a site comprising Perth sand. The tests were performed at different times corresponding to the end of a wet season and end of a dry season. Some tests were performed on parts of the site near large trees, while other tests were performed in an open area. Figures 2.10 and 2.11 present
the results of tests conducted in the treed area and open area, respectively. Near the trees it was found that, at the end of the dry season, $q_c$ was significantly higher than the corresponding values at the end of the wet season. Also, in the open area, the seasonal change had a very minor influence on the test results. The main conclusion drawn in the Lehane et al. (2004) investigation was that when degree of saturation ($S_r$) is less than about 0.1 suction is large enough to have a significant effect on $q_c$, although the presence of tree roots was required to cause $S_r$ to drop below 0.1 and therefore increase suction significantly. This latter point was evidenced by the results in the open area which were virtually indistinguishable at the ends of the wet and dry seasons.

Tan (2005) conducted a series of miniature cone penetration tests in unsaturated Minco silt within a calibration chamber. Test beds were prepared to encompass suction values ranging from 15 kPa to 60 kPa. The suction was determined directly via tensiometers and indirectly by using the soil-water characteristic curve and moisture content ($w$) of specimens. The specimens were subjected to a constant isotropic net confining stress of 103 kPa. It was observed that suction noticeably influences cone penetration resistance for the range of suction values considered. However, no tests were performed in saturated state and therefore comparison and correlating between results of unsaturated and saturated conditions have not been possible.

2.9 CONCLUDING REMARKS

Unsaturated soils are widely encountered and the concept of effective stress provides an efficient way to describe their stress-strain behavior and to investigate effects of suction on engineering properties.

The cone penetration test (CPT), a commonly used in situ test, can provide less costly and more rapid characterization of the unsaturated soil properties. There is evidence, that suction significantly affects CPT results (Hryciw and Dowding, 1987; Lehane et al., 2004; Russell and Khalili, 2006b). However, no methods are currently available for interpreting the CPT in unsaturated soils.
Calibration chambers have been used in the past to develop correlations between CPT results and soil properties under laboratory controlled conditions for saturated or dry soils. To study the CPT in unsaturated soils a new stress and suction controlled calibration chamber is needed overcoming problems of others.

An appropriate theoretical tool for interpreting CPT results is cavity expansion theory. However, careful account of the specimens finite size and boundary conditions is needed while applying the theory to interpretation of the CPT results in a calibration chamber and this has not been fully resolved in the literature.
Figure 2.1 Unsaturated soils consist of three phases: solid particles, air and water occupying the pore space.
Figure 2.2 A general soil water characteristic curve for a drying path showing the three identifiable zones (after Fredlund et al., 2011).

Figure 2.3 Typical soil water characteristic curves for different soil types (after Vanapali et al., 1999).
Figure 2.4 Detailed features of a piezocone penetrometer (after Lunne et al., 1997).
Figure 2.5 Pore water pressure effects on measured parameters in cone penetration tests (after Lunne et al., 1997).

Figure 2.6 Calibration chamber size effects on cone resistance for Hokksund sand, $\sigma'v = 50$ kPa (after Parkin and Lunne, 1982).
2.7 Effect of compressibility on cone resistance ($q_c$), effective vertical stress ($\sigma'_v$) and relative density ($D_r$) correlations (after Robertson and Campanella, 1983a).

2.8 Normalised cone resistance versus state parameter for different sands (after Been et al., 1986).
Figure 2.9 Relationship between normalized cone resistance and friction angle (after Robertson and Campanella, 1983a).
Figure 2.10 In-situ cone penetration resistance for Perth sand in a treed area at the ends of wet season (represented by solid symbols) and dry season (represented by hollow symbols) (after Lehane et al., 2004).

Figure 2.11 In-situ cone penetration resistance for Perth sand in an open area at the ends of wet season (represented by solid symbols) and dry season (represented by hollow symbols) (after Lehane et al., 2004).
CHAPTER 3

DEVELOPMENT OF A NEW CALIBRATION CHAMBER FOR UNSATURATED SOILS

3.1 INTRODUCTION
This chapter presents details of a new suction and stress controlled calibration chamber for conducting cone penetration tests in variably saturated soils (Pournaghiazar et al., 2011a). The equipment was designed and manufactured as part of this investigation. The design of the equipment and its details are explained in section 3.2. The control and measurement devices are explained in section 3.3 and the testing setup is discussed in section 3.4.

3.2 THE NEW DESIGN
The new design of the calibration chamber utilizes some of the features of previous chambers (Bellotti et al., 1982; Sweeney and Clough, 1990; Anderson et al., 1991; Miller et al., 2002) but key improvements have been incorporated, including a novel specimen formation system, a modified axial load application system, and the use of
an enhanced axis translation technique for application of suction within the system. Moreover, specimens of different soil types (cohesive or granular) can be prepared adding to the versatility of the chamber. Both constant-stress and no-displacement conditions can be imposed at the specimen boundaries. Lateral confining pressure is applied by water pressure acting on a rubber membrane enclosing the soil specimen. Vertical pressure is applied by a hydraulic loading ram pushing on the chamber piston connected to the base of the specimen. Suction in the specimen is controlled using the axis translation technique. The chamber can accommodate cylindrical specimens with a height of 840 mm and diameter of 460 mm. These dimensions have been chosen to minimize lateral, top and bottom boundary effects considering an optimum cost of the manufacturing and so that the rubber membrane, which could only be manufactured to a limited number of sizes, fitted snugly around the end plates.

The new design consists of twelve main components: i) a stainless steel chamber shell, ii) a stainless steel moveable former, iii) a piston, iv) cadmium-plated steel wings and bottom ring, v) cadmium-plated steel flanges, vi) a cadmium-plated steel base plate, vii) a rubber membrane, viii) a stainless steel bottom plate and porous disks, ix) a loading ram, x) a stainless steel top plate, xi) stainless steel top cap, and xii) control and measurement units. A photograph and cross section of the device are shown in Figures 3.1 and 3.2, respectively. The design drawings of the equipment and details of different segments (for manufacturing) are presented in Appendix A.

3.2.1 Chamber shell and flanges
The chamber shell is made of stainless steel with an inner diameter of 690 mm, wall thickness of 10 mm and height of 950 mm. Two steel flanges of 710 mm inner diameter, 850 mm outside diameter and 30 mm thickness are welded to both ends of the shell. The shell is attached to the top cap and bottom ring using bolts through the flanges. Rubber o-rings placed in specially machined grooves between the flanges and the top cap and the bottom ring provide water tight connections under pressure. The chamber is designed for a safe working pressure of 2000 kPa.
3.2.2 Moveable former

The specimen former consists of four stainless steel cylinder quarters with each quarter comprising two segments joined at the mid-height of the former. The mould quarters are split at mid-height to facilitate assembly of the segments after manufacturing. Two handles are attached to each cylinder quarter near the top and bottom; this enables manual movement of the quarters towards and away from the center of the chamber. During specimen preparation, the quarters of the former are pushed together (Figure 3.3a) and locked into position to form a rigid cylindrical mould. A one piece collar connected to the top flange of the quarters when they are pushed together adds to the rigidity, as do eight support rods that pass through the collar along the full height of the former and into the chamber ring. The rigid mould has an inside diameter of 476 mm, height of 950 mm and wall thickness of 5 mm. Loose soil placed in the mould can be subjected to vertical compaction and compression to form a specimen without experiencing problems of bulging or distortion.

After specimen preparation, the chamber is assembled and a confining cell pressure is applied. The quarters of the former may then be pulled away from the specimen, as shown in Figure 3.3b, to allow a constant-stress condition at the specimen boundary. The surfaces of the former quarters are perforated to ease their movement when a confining cell pressure is applied.

3.2.3 Chamber piston

The piston consists of a 50 mm thick and 330 mm diameter steel plate connected to a hollow cylinder skirt. The skirt is guided as it moves vertically up and down by sliding against eight durable plastic sleeves. Each sleeve is attached to a metal segment and chamber wing.

The piston rests on a hydraulic loading ram. There are two access holes in the skirt walls to allow passage of hydraulic tubes to the inside of the piston where the loading ram is located.

A steel ring of 850 mm outside diameter, 330 mm inside diameter and 40 mm thickness welded to the top of the eight steel wings supports the chamber body. The
steel wings are welded to the chamber base plate every 45°. A heavy duty piston seal is fitted inside a groove cut in to the inner perimeter of the steel ring. The piston seal prevents the pressurized cell fluid passing through the base of the chamber.

3.2.4 Specimen bottom plate

The stainless steel specimen bottom plate is 50 mm thick and has a diameter of 460 mm. It is attached to the chamber piston through equally spaced bolts. Eight high air entry value porous ceramic disks are embedded in the upper face of the bottom plate, which enables control and maintenance of suction in the soil specimen using the axis translation technique (Hilf, 1956). The ceramic disks used in this chamber are 10.3 mm thick, 103 mm in diameter and have an air entry value of 500 kPa.

Each porous disk sits snugly in a specially machined cavity. A curved groove is cut into the base of each cavity enabling the pore water passing through the discs to be drained from the system. The curved grooves also serve as channels for flushing air bubbles that may become trapped or accumulated as a result of air diffusion. A hole drilled at one end of each groove is connected to a bleeding valve positioned underneath the bottom plate. A hole at the other end is connected to the pore pressure measurement and control system. Figure 3.4 shows the cavities for each disc, the curved grooves and the ceramic discs sitting in the cavities. The discs are held in position using an epoxy resin to bond their circumference to the inner circumference of each cavity. The upper surfaces of the discs are level with the surface of the bottom plate.

The high air entry disks must be saturated prior to testing. To achieve this, de-aired water is forced to pass through the ceramic disks by applying a pore water pressure of 25 kPa underneath the disks. Water flows upwards and exits the surface at atmospheric pressure. The procedure is continued for at least 3 days and the volume of water flow is measured to make sure that there has been sufficient flow for the disks to become saturated. Applying pressures more than 25 kPa may destroy the seal around the disks created by epoxy resin. After application of suction by using the axis translation technique any diffused air bubbles trapped in the grooves (underneath the ceramic
discs) was were monitored which ensured effectiveness of procedure and flushed out as necessary to ensure the water pressure reading was accurate.

Note that saturation of the high air entry value discs could have been alternatively performed in a different way by pressurizing water inside the cell (in the absence of the specimen) to flush water through the ceramic discs top to bottom which would requires an enormous volume of de-aired water and a time consuming extra procedure of assembling and dismantling of the calibration chamber. For practical purposes the former method of saturation has been adopted and is a standard method used in research equipment having fixed in-place high air entry ceramics.

### 3.2.5 Specimen top plate and chamber top cap

The stainless steel top plate is 50 mm thick and has a diameter of 460 mm. A 50 mm diameter hole cut through the centre of the top plate allows the cone penetrometer to be pushed into the specimen. There are four threaded holes cut through the top plate: two are connected to the pore air pressure control system and two are connected to either a pore water pressure control system or used as an air vent during specimen flushing. Four bolts fasten the specimen top plate to the top cap. Several o-rings placed in grooves machined into the top plate near the circumference of the top plate, central hole and threaded holes are used to create water tight seals.

The stainless steel top cap is 30 mm thick and has a diameter of 850 mm. Like for the specimen top plate, a hole of 50 mm in diameter is cut through the centre of the top cap allowing the penetrometer to be pushed into the specimen. Four threaded holes are aligned with holes on the top plate, two that connect to the pore air pressure control system and two that connect to either a pore water pressure control system or are used as a vent during specimen flushing. The top cap is joined to the top flange using twelve bolts.

### 3.2.6 Specimen membrane

The specimen membrane is made of natural and superior abrasion resistant rubber lining, manufactured by Q & R Industrial Hoses Pty Ltd. It is cylindrical shape, 8 mm thick and 476 mm in outside diameter. The membrane is connected to the top and bottom plate by hose clamps which also form water tight seals. Specimen formation
included first fastening the rubber membrane to the bottom plate using a 25 mm wide hose clamp. Once the membrane was in place the chamber shell was put into position and the four sections of the former were moved towards the centre of the chamber where they were fixed and the collar fastened.

A 50 mm thick gravel layer was placed on top of the specimen to enable even spread of air/water pressure. The top plate was then placed in position and the rubber membrane clamped to the top plate. The chamber top cap was fastened to the top plate and chamber shell.

3.3 CONTROL AND MEASUREMENT DEVICES

The calibration chamber is fitted with a number of control and measurement devices to record the applied cell pressure, pore water pressure, pore air pressure and vertical stress along with displacement of the specimen base and volume changes of the cell water and pore water. The general arrangement of the control system is shown in Figure 3.5.

3.3.1 Cell pressure control and measurement

Cell water pressure is supplied using laboratory air pressure connected to an air-water interface cylinder. The cell pressure can be adjusted using an air pressure regulator connected to the air-water interface cylinder. Analogue pressure gauges installed on the control panel are used to measure pressures. Coarse cell volume change is measured using the water level inside the air-water interface cylinder viewed through a graduated transparent plastic tube positioned outside the cylinder (accurate to about ±10 mL). Finer volume change measurements are made using a glass burette and water-oil interface (accurate to about ±0.5 mL).

3.3.2 Pore water and pore air pressure control and measurement

Two separate lines of pore water are pressurized using air-water interface cylinders connected to laboratory air pressure and air pressure regulators. One line enables application of pore water pressure to the specimen through perforated copper tubes at the bottom plate (as can be seen in Figure 3.4b). The copper tubes also enable a sand
specimen to be flooded with water quickly and also provide control of pore water pressure for tests on saturated specimens. The other line, connected to the high air entry value disks embedded in the bottom plate, controls pore water pressure for the unsaturated tests. Pore water volume changes are measured using separate volume change units comprising glass burettes and water-oil interfaces.

Pore air pressure is supplied directly to the top of a specimen using laboratory air pressure and an air pressure regulator. It is measured using a pressure gauge installed on a control board.

3.3.3 Vertical stress application and control

For application and maintenance of vertical stress on the specimen a hydraulic ram pushes upwards on the piston and bottom plate. It is a spring return 10 ton ram (Model 102) manufactured by Simplex Cylinders. An electric micro pump of model GMP-08-120 manufactured by Riken Kiki is used for operation of the hydraulic ram. The pump is fitted with a DPS700A pressure switch, a manual hand pump, secondary reservoir and an analogue pressure gauge.

A linear varying displacement transducer (LVDT), mounted below the piston, tracks vertical displacements of the ram (which are equal to vertical deformations of the specimen) and is logged using a computerized system. In addition to the vertical force supplied by the hydraulic ram, cell water pressure pushing upward on the specimen bottom plate induces another vertical force and this must be considered in determining the total vertical force and stress on the specimen.

3.4 TEST SETUP AND CALIBRATION OF THE PENETROMETER

Cone penetration tests were conducted using a miniature electrical cone, manufactured by A.P van den Berg (model ELC2). The cone had a diameter of 16 mm, cone tip area of 2 cm$^2$ and and friction sleeve area of 30 cm$^2$. 

3-7
The cone was pushed using a HYSON 100 kN single cylinder static cone penetrometer, powered by a petrol driven power pack. A loading frame, specially built to mount the HYSON penetrometer, was positioned above the chamber and bolted to the top cap and top flange. The nominal and maximum load capacities of the cone were 50 MPa and 100 MPa, respectively. Figure 3.6 shows the frame and penetrometer mounted on the top of the chamber.

The cone can be pushed into the soil at constant rates ranging from 0.2 to 2 cm/s. The data recorded during testing may include cone penetration resistance $q_c$, sleeve friction $f_s$, pore water pressure $u_w$ (for saturated tests only) and inclination $I$.

The cone penetrometer was calibrated by the manufacturer (Van den Berg) immediately prior to the tests being conducted. The calibration data is stored in the data file cone.dat prepared by the manufacturer which the software Gorilla reads. The calibration certificate, data file and technical manual for Gorilla have been presented in Appendix B.

3.4.1 Bush cylinder

During specimen consolidation and the application of suction a specially designed hollow bush cylinder was used to create a seal around the cone and the centre hole of the chamber top cap prior to penetration. A photograph of the bush cylinder mounted on top of the chamber is shown in Figure 3.7. Detailed drawings of the bush cylinder is presented in Appendix A.

The bush cylinder consisted of a 76.2 mm inner diameter, 220 mm high and 6 mm thick brass tube welded to a hollow base plate of 204 mm diameter and 10 mm thickness. Four bolts fastened the base plate to the chamber top cap and an o-ring placed in a groove near the circumference of the opening provided a water tight seal. The top of the cylinder comprised a flange 20 mm wide and 10 mm thick which was bolted to a circular plate with a diameter of 128 mm and thickness of 15 mm. An o-ring placed between the plate and flange formed a water tight seal. The 16 mm diameter hole positioned in the centre of the plate enabled pushing of the cone through the bush cylinder. The gap between the circumference of the hole and extension rods
attached to the cone was sealed using two rubber o-rings placed in specially machined grooves.

3.5 CONCLUDING REMARKS

A calibration chamber suitable for conducting cone penetration tests in dry, saturated and unsaturated soils is presented. The novel aspects of the design include the specimen formation system, a modified axial load application system, and the use of an enhanced axis translation technique for application, measurement and control of suction within the system. Incorporation of a moveable former into design of the chamber resolves problems associated with specimen preparation and prevents disturbance of specimens. Moreover, specimens of different soil types (cohesive or granular) can be prepared adding to the versatility of the chamber.

The calibration chamber is fitted with a number of control and measurement devices to record the cell pressure, pore water pressure, pore air pressure, vertical stress and displacement.

Cone penetration tests were conducted using a miniature electrical cone having a diameter of 16 mm. A loading frame was specially built to mount the penetrometer and positioned above the chamber prior to testing. A specifically designed hollow bush cylinder was used to create a seal around the cone and the centre hole of the chamber top cap during specimen consolidation and penetration.

The calibration chamber was manufactured, assembled and operated successfully and results of laboratory controlled CPTs conducted in dry, saturated and unsaturated sand within the calibration chamber are presented in chapter 4.
Figure 3.1 Photograph of the calibration chamber.
Figure 3.2 Calibration chamber cross section.
Figure 3.3 Plan view of movable former, a) specimen preparation configuration, b) testing configuration.
Figure 3.4 Specimen bottom plate: a) Curved grooves; b) High air entry value porous disks embedded in the plate and copper tubing for saturation of specimen.
Figure 3.5 The calibration chamber control system.
Figure 3.6 Penetrometer attached to the frame and bolted to the top of the calibration chamber.
3.7 The bush cylinder used for the sealing around the cone and the centre hole of the chamber top cap.
CHAPTER 4

EXPERIMENTAL INVESTIGATION OF SUCTION EFFECTS ON CONE PENETRATION TEST RESULTS IN SAND

4.1 INTRODUCTION

To investigate the cone penetration test in unsaturated sands an extensive range of laboratory controlled CPTs were conducted within the calibration chamber presented in chapter 3. Specifically, the experimental program was designed to study how suction affects cone resistance for a range of isotropic confining net stresses, initial controlled suctions and initial relative densities.

In this chapter, firstly details of the test material are presented. Secondly, in regard to calibration chamber testing, the specimen formation methods and testing procedures are explained and discussed. Finally, results of CPTs performed in saturated and unsaturated sand specimens are presented and the influence of suction on the cone penetration resistance is highlighted. The contribution of suction to the penetration
resistance and its significance for different stress levels and initial conditions is also discussed.

**4.2 TEST MATERIAL**

A predominantly quartz sand was used in all the experiments. The sand was sourced from the dunes around Sydney, Australia, and hereafter will be referred to as Sydney sand.

The particle size distribution for Sydney sand is shown in Figure 4.1. Grain diameters at 60% passing ($D_{60}$), 50% passing ($D_{50}$), 33% passing ($D_{30}$) and 10% passing ($D_{10}$) are 0.33mm, 0.31mm, 0.23mm and 0.18mm respectively. These correspond to a uniformity coefficient ($C_u$) of 1.83 and a coefficient of curvature ($C_c$) of 0.89. The sand was classified as poorly graded (SP), medium coarse grained according to the Unified Soil Classification System. As reported by Russell (2004), a particle density of $G_s = 2.65$ g/cm$^3$ applies to Sydney sand. The minimum dry density ($\rho_{min}$) is 1.38 g/cm$^3$ corresponding to a maximum void ratio of $e_{max} = 0.92$. Also the maximum dry density ($\rho_{max}$) is $1.66$ g/cm$^3$ corresponding to a minimum void ratio of $e_{min} = 0.6$ (Russell, 2004). An extensive experimental program was conducted by Russell (2004) to characterize the stress strain behavior of saturated and unsaturated Sydney sand. Certain features of the experimental investigations undertaken by Russell (2004) are presented in Appendix C. They include soil water characteristic curves (SWCCs) and the mechanical stress-strain behaviour observed in saturated and unsaturated triaxial tests.

**4.3 CONE PENETRATION TESTS**

Numerous cone penetration tests have been conducted on saturated, dry and unsaturated Sydney sand within the calibration chamber to study the effect of suction on the cone resistance. The tested specimens had initial relative densities of $D_r = 61\%$ and 33% corresponding to initial specific volumes of $v_0 = 1.723$ and 1.813, respectively, and were subjected to a range of confining net stresses and suctions.
4.3.1 Specimen formation

Specimen formation included first fastening the rubber membrane to the bottom plate using a 25 mm wide hose clamp. The membrane utilized was made of natural and abrasion resistant rubber. It had a wall thickness of 8 mm and inside diameter of 460 mm. Once the membrane was in place the chamber shell was put into position and the four sections of the former were moved towards the centre of the chamber where they were fixed and the collar fastened.

Dry sand specimens were prepared by the pluvial deposition technique (Jacobsen, 1976). Sand was placed in a large hopper positioned above the calibration chamber (Figure 4.2) and allowed to flow through the base of the hopper, then through a diffuser, before falling into the chamber (Figure 4.3). The flow rate was controlled by adjusting the size of the opening at the base of the hopper. The diffuser was made of two sieves connected to a pulley system which was used to adjust the drop height. A uniform flow rate of sand through the diffuser and a uniform drop height (distance between diffuser and placed sand surface) enabled preparation of homogenous specimens.

Decreasing the flow rate and increasing the drop height increased the density (Rad and Tumay, 1987; Lo Peresti et al., 1992). By conducting several trials it was confirmed that this system was able to create specimens with repeatable densities equal to target values dependant on the drop height and controlled flow rate. To achieve a relative density of $D_r = 61\%$ the flow rate and drop height were adjusted to 42g/s and 0.7m, respectively. A relative density of $D_r = 33\%$ was achieved by adjusting the flow rate and drop height to 66g/s and 0.25m, respectively.

A 50 mm thick gravel layer was placed on top of the specimen to enable even spread of air/water pressure (Figure 4.4). A PVC pipe of 100 mm diameter and 50 mm long was placed in the gravel layer (aligned with the top plate centre hole) and a sand layer was placed inside the pipe section that enables cone to be pushed into the specimen without touching the gravel layer. The top plate was then placed in position and the
rubber membrane clamped to the top plate (Figure 4.5). The chamber top cap was fastened to the top plate and chamber shell.

A small confining pressure was applied to the specimen and the moveable sides of the former were pulled away from the specimen towards the chamber shell. The cell pressure and axial pressure were then increased to the desired values and the specimen was allowed to consolidate. The volume change of the specimen was measured during this process to determine when consolidation was complete. Specimen saturation was achieved by passing de-aired distilled water through the perforated copper tubes located at the bottom of the specimen while applying a 10 kPa vacuum to the top of the specimen.

Unsaturated specimens were formed by first saturating the specimens and then letting the moisture content reduce to achieve a target suction. As will be discussed in section 4.37 in order to achieve an even distribution of moisture content (uniform suction) throughout the specimen this method of specimen formation has been adopted. Initially, water was allowed to drain freely under its self weight by opening the valves connected to the copper tubing at the base. Once the drainage of water was completed, the copper tubing was sealed, and pore air pressure and pore water pressure were applied via the top and through the high air entry ceramic discs at the base, respectively.

The pore water volume change was monitored during this process until moisture equilibrium was reached within the specimen. The equilibration time is 7 days for the applied suction of 200 kPa and 19 days for the applied suction of 50 kPa. Post-test moisture content analyses (Appendix D) indicated that this method lead to an even moisture distribution throughout a specimen. For example, the moisture content values ranging from 1.5% to 1.9% were recorded throughout the specimen after tests subjected to a constant suction of 200 kPa. The post test moisture content values for applied suctions of 50 kPa and 25 kPa are ranging from 2.8% to 3% and 4.5 to 4.9% respectively. In accordance with the applied suction on the specimens, these moisture content values are in good agreement with soil water characteristic curves (SWCCs) presented in Appendix C.
4.3.2 Testing procedure
Cone penetration tests were conducted using a miniature electrical cone (Figure 4.6). The cone had a 16 mm diameter and the cone tip area was 2 cm². All the tests were conducted with the standard penetration rate of 20 mm/s.

As mentioned previously, for unsaturated testing, matric suction was applied to the specimens by increasing the air pressure connected to the chamber top cap while maintaining constant pore water pressure applied at the specimen base through high air entry disks. In trial tests, it was observed that connecting laboratory air pressure directly to the top cap fittings resulted in local drying of the specimen surface, which extended as far as 0.2 m below the top. This was caused by pore water evaporating into the dry air. Therefore, to prevent localized drying, the laboratory air was passed through a water bath cylinder before entering the chamber as shown in Figure 4.7. The issue of localized drying was successfully resolved by using this method.

4.3.3 Results
As mentioned in section 4.3 a comprehensive program of CPTs have been conducted on saturated, dry and unsaturated Sydney sand within the calibration chamber to study the effect of suction on the cone resistance. The tested specimens had initial relative densities of \( D_r = 61\% \) and 33\% corresponding to initial specific volumes of \( v_0 = 1.723 \) and 1.813, respectively, and were subjected to a range of confining net stresses (25 kPa, 50 kPa, 100 kPa and 150 kPa) and suction values ranging from 7 kPa to 200 kPa.

Figures 4.8 to 4.10 present plots of cone resistance versus depth for CPTs performed in saturated specimens with an initial relative density of \( D_r = 33\% \) \( (v_0 = 1.813) \). The tests were conducted at constant isotropic effective confining stresses of 25 kPa, 50 kPa and 100 kPa and are referred to using the symbols SL25, SL50 and SL100, respectively. The test results for CPTs performed in saturated specimens with an initial relative density of \( D_r = 61\% \) \( (v_0 = 1.723) \) are presented in Figures 4.11 to 4.14. The tests were conducted at constant isotropic effective confining stresses of 30 kPa, 50 kPa, 100 kPa and 150 kPa and are referred to using the symbols SM30, SM50, SM100 and SM150, respectively.
Figure 4.15 presents a plot of cone resistance versus depth for a CPT performed in an unsaturated specimen with an initial relative density of $D_r = 33\%$ and constant isotropic net confining stress of 25 kPa subjected to a suction varying from 13 kPa at the top of the specimen to 7 kPa at a depth of 0.6 m. The test is referred to using a symbol UL25-7-13. This test was performed to specifically investigate the influences of small suctions on the cone resistance. These small suctions were induced using a special procedure as now described. After completion of specimen saturation, a reservoir was connected to the line supplying water to the perforated copper tubes on the bottom plate. The reservoir was then placed on the laboratory floor adjacent to the chamber, so that the water level within the reservoir was 1.3m below the specimen top. Water was then allowed to drain from the specimen through the reservoir while the water level within the reservoir was kept constant. The suctions induced were equal to or slightly larger than the air entry value of 7kPa. Given that gravity is always present, then the same suction variation across the specimens occurs in all tests but the effect is negligible in those tests where high suction levels are applied.

The results from two further CPTs performed in unsaturated specimens with an initial relative density of $D_r = 33\%$ and constant isotropic net confining stress of 25 kPa are presented in Figures 4.16 and 4.17. The specimens were subjected to initial controlled suctions of 50 kPa and 200 kPa and are referred to using the symbols UL25-50 and UL25-200, respectively.

Figures 4.18 and 4.19 present the test results for CPTs performed in unsaturated specimens with an initial relative density of $D_r = 33\%$ and constant isotropic net confining stress of 50 kPa. The specimens were subjected to initial controlled suctions of 25 kPa and 200 kPa and are referred to using the symbols UL50-25 and UL50-200, respectively.

The test results for CPTs performed in unsaturated specimens with an initial relative density of $D_r = 33\%$ and constant isotropic net confining stress of 100 kPa are presented in Figures 4.20 and 4.21. The specimens were subjected to initial controlled suctions of 25 kPa and 200 kPa and are referred to using the symbols UL100-25 and UL100-200, respectively.
Figures 4.22 and 4.23 present the test results for CPTs performed in unsaturated specimens with an initial relative density of $D_r = 61\%$ and constant isotropic net confining stress of 30 kPa. The specimens were subjected to initial controlled suctions of 25 kPa and 200 kPa and are referred to using the symbols UM30-25 and UM30-200, respectively.

The test results for CPTs performed in unsaturated specimens with an initial relative density of $D_r = 61\%$ and constant isotropic net confining stress of 50 kPa are presented in Figures 4.24 and 4.25. The specimens were subjected to initial controlled suctions of 25 kPa and 200 kPa and are referred to using the symbols UM50-25 and UM50-200, respectively.

Figures 4.26 and 4.27 present the test results for CPTs performed in unsaturated specimens with an initial relative density of $D_r = 61\%$ and constant isotropic net confining stress of 100 kPa. The specimens were subjected to initial controlled suctions of 25 kPa and 200 kPa and are referred to using the symbols UM100-25 and UM100-200, respectively.

Figure 4.28 presents a plot of cone resistance versus depth for a CPT performed in an unsaturated specimen with an initial relative density of $D_r = 61\%$ and a constant isotropic net confining stress of 150 kPa subjected to an initial controlled suction of 200 kPa. The test is referred to using the symbol UM150-200.

It is observed that the $q_c$ values gradually increased in the upper 0.1 m of the specimen, before peaking and then reducing slightly to a relatively constant value at a depth of around 0.25 m. The peaks recorded at about 0.1 m are due to the interaction of the cone induced zones of plasticity with the rigid top plate. The gravel layer placed on top of the specimen may also have an influence on the results in upper portion of the plots (up to a depth of 0.25 m). The thickness of the gravel layer was not identical for all tests due to difficulties of placing top plate and leveling it with chamber flanges and it is believed that the observed slight variation in locations of peaks is due to this fact. From about 0.55 m the $q_c$ values show a modest increase from the constant values due to interaction with the rigid base boundary. Similar observations have also been reported by Parkin and Lunne (1982), Houlsby and Hitchman (1988) and Sweeney and
Clough (1990). The relevant portion of the test records that are used for calibration purposes is therefore between the depths of 0.3 m and 0.55 m where the $q_c$ values are approximately constant and free from the influence of the top and bottom boundaries.

The average $q_c$ values for the CPTs conducted in saturated and unsaturated sand specimens with an initial relative density of $D_r = 33\%$ are summarized in Table 4.1. Note that the average $q_c$ values were calculated from recorded data between the depths of 0.3 m and 0.55 m for each test. Also in calculation of the average values, minor non-uniformities observed in some data were omitted.

The average $q_c$ values for the CPTs conducted in saturated and unsaturated sand specimens with an initial relative density of $D_r = 61\%$ are summarized in Table 4.2. The rigid bottom boundary influenced these particular results from a depth of 0.5 m and therefore the average $q_c$ values for these tests were calculated from recorded data between the depths of 0.3 m and 0.5 m.

Figures 4.29 and 4.30 present plots of cone resistance versus depth for CPTs performed in dry specimens with an initial relative density of $D_r = 61\%$ subjected to constant isotropic confining stresses of 50 kPa and 100 kPa and are referred to by symbols DM50 and DM100, respectively. These CPT results along with those from saturated specimens (with identical relative density and corresponding isotropic effective confining stress) have been plotted again in Figures 4.31 and 4.32 for comparison. It is clear that there is negligible difference between average cone resistance values obtained from tests in purely dry and fully saturated specimens.
Table 4.1 Average cone resistance ($q_c$) values (calculated between the depths of 0.3m and 0.55m) for the CPTs conducted in saturated and unsaturated sand specimens with an initial relative density of $D_r = 33\%$.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Isotropic net confining stress (kPa)</th>
<th>Suction (kPa)</th>
<th>Average $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL25</td>
<td>25.0</td>
<td>0.0</td>
<td>2.0</td>
</tr>
<tr>
<td>UL25-7-13</td>
<td>25.0</td>
<td>10.0 (Average)</td>
<td>2.7</td>
</tr>
<tr>
<td>UL25-50</td>
<td>25.0</td>
<td>50.0</td>
<td>3.4</td>
</tr>
<tr>
<td>UL25-200</td>
<td>25.0</td>
<td>200.0</td>
<td>3.8</td>
</tr>
<tr>
<td>SL50</td>
<td>50.0</td>
<td>0.0</td>
<td>3.4</td>
</tr>
<tr>
<td>UL50-25</td>
<td>50.0</td>
<td>25</td>
<td>4.2</td>
</tr>
<tr>
<td>UL50-200</td>
<td>50.0</td>
<td>200</td>
<td>5.1</td>
</tr>
<tr>
<td>SL100</td>
<td>100.0</td>
<td>0.0</td>
<td>5.8</td>
</tr>
<tr>
<td>UL100-25</td>
<td>100.0</td>
<td>25.0</td>
<td>6.6</td>
</tr>
<tr>
<td>UL100-200</td>
<td>100.0</td>
<td>200.0</td>
<td>7.6</td>
</tr>
</tbody>
</table>

4.3.4 Repeatability of test results

A total of 4 CPTs were repeated to confirm repeatability of test results and the specimen formation methods adopted. Figure 4.33 shows a repeated CPT in a saturated sand specimen with an initial relative density of $D_r = 33\%$ subjected to an isotropic effective confining stress of 50 kPa (referred to as SL50R). A plot of a repeated CPT for the unsaturated specimen with initial relative density of $D_r = 33\%$ and isotropic net confining stress of 50 kPa subjected to initial controlled suction of 200 kPa is shown in Figure 4.34 (referred to as UL50-200R). Figures 4.35 and 4.36 show repeated CPTs in unsaturated specimens with an isotropic net confining stress of 100 kPa, initial suction
of 25 kPa having relative densities of $D_r = 33\%, 61\%$ (referred to as UL100-25R and UM100-25R, respectively). The average cone resistance values for repeated tests are summarized in Table 4.3.

**Table 4.2** Average cone resistance ($q_c$) values (calculated between the depths of 0.3m and 0.5m) for the CPTs conducted in saturated and unsaturated sand specimens with an initial relative density of $D_r = 61\%$.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Isotropic net confining stress (kPa)</th>
<th>Suction (kPa)</th>
<th>Average $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SM30</td>
<td>30.0</td>
<td>0.0</td>
<td>3.1</td>
</tr>
<tr>
<td>UM30-25</td>
<td>30.0</td>
<td>25.0</td>
<td>3.9</td>
</tr>
<tr>
<td>UM30-200</td>
<td>30.0</td>
<td>200.0</td>
<td>4.8</td>
</tr>
<tr>
<td>SM50</td>
<td>50.0</td>
<td>0.0</td>
<td>5.0</td>
</tr>
<tr>
<td>UM50-25</td>
<td>50.0</td>
<td>25</td>
<td>5.8</td>
</tr>
<tr>
<td>UM50-200</td>
<td>50.0</td>
<td>200</td>
<td>6.7</td>
</tr>
<tr>
<td>SM100</td>
<td>100.0</td>
<td>0.0</td>
<td>11.0</td>
</tr>
<tr>
<td>UM100-25</td>
<td>100.0</td>
<td>25.0</td>
<td>12.2</td>
</tr>
<tr>
<td>UM100-200</td>
<td>100.0</td>
<td>200.0</td>
<td>13.7</td>
</tr>
<tr>
<td>SM150</td>
<td>150.0</td>
<td>0.0</td>
<td>18.0</td>
</tr>
<tr>
<td>UM150-200</td>
<td>150.0</td>
<td>200.0</td>
<td>20.2</td>
</tr>
</tbody>
</table>

Results of the repeated and original tests are replotted in Figures 4.37 to 4.40 confirming very good repeatability of the testing procedure and specimen formation method adopted. Also the observed similarity between test results obtained from dry
and saturated specimens with identical conditions adds to the evidence supporting reliability of the results. Comparing tabulated values of the average cone resistance from Tables 4.3 with corresponding values in Tables 4.2 and 4.1 reveals a variance of less than 0.2 MPa which is acceptable for calibration chamber testing (Lunne et al., 1997).

Table 4.3 Average cone resistance ($q_c$) values for the repeated CPTs in saturated and unsaturated sand specimens.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Relative density ($D_r$)</th>
<th>Isotropic net confining stress (kPa)</th>
<th>Suction (kPa)</th>
<th>Average $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL50R</td>
<td>33%</td>
<td>50</td>
<td>0.0</td>
<td>3.3</td>
</tr>
<tr>
<td>UL50-200R</td>
<td>33%</td>
<td>50</td>
<td>200.0</td>
<td>5.3</td>
</tr>
<tr>
<td>UL100-25R</td>
<td>33%</td>
<td>100</td>
<td>25.0</td>
<td>6.5</td>
</tr>
<tr>
<td>UM100-25R</td>
<td>61%</td>
<td>100</td>
<td>25.0</td>
<td>12.0</td>
</tr>
</tbody>
</table>

4.3.5 Discussion

The significant effect of suction on the cone penetration resistance for different sand specimens subjected to a range of isotropic confining net stresses is shown in Figures 4.41 to 4.47.

More specifically, Figure 4.41 highlights the effect of suction on the cone resistance for sand specimens having an initial relative densities of $D_r$ =33% and subjected to isotropic net confining stresses of 25 kPa. A broad range of suctions were applied to the specimens which enables a comprehensive study of the suction influence, with suction values starting near the air entry value of 7kPa and increasing to 200 kPa. Inducing suction by allowing the pore water to drain under its self weight for UL25-7-13 resulted in suction varying from 10 kPa to 7.5 kPa between depths of 0.3 and 0.55
m, respectively, and the 2.5 kPa differential apparently did not significantly alter the cone resistance measured within that depth range. However, the suction did noticeably increase cone resistance from the value measured in a saturated test SL25 (when suction was zero). Suctions of 50 kPa and 200 kPa increased the average cone resistance values by 70% and 90% above that measured in the saturated test.

The observed increase in the cone resistance is due to an increase in the effective stress as well as the stiffening of the soil skeleton due to matric suction in the specimens prior to testing.

Effects of suction on the cone resistance values for a sand specimen with an initial relative density of $D_r = 33\%$ subjected to an isotropic net confining stress of 50 kPa are highlighted in Figure 4.42. It is observed that suctions of 25 kPa and 200 kPa increased the average cone resistance values by about 24% and 50%, respectively. Figure 4.43 show that for a sand specimen with an initial relative density of $D_r = 33\%$ subjected to isotropic net confining stress of 100 kPa suctions of 25 kPa and 200 kPa increased the average cone resistance values by about 14% and 31%, respectively.

The same trend is observed for the CPTs performed in specimens with an initial relative density of $D_r = 61\%$. Figure 4.44 shows that for a specimen subjected to a constant isotropic net confining stress of 30 kPa, suctions of 25 kPa and 200 kPa increase the average cone resistance values by 26% and 55%, respectively. For a constant isotropic net confining stress of 50 kPa, suctions of 25 kPa to 200 kPa increase the average cone resistance values by 16% to 34% (Figure 4.45). In sand specimens subjected to a constant isotropic net confining stress of 100 kPa, suctions of 25 kPa to 200 kPa increase the average cone resistance values by 11% to 25% (Figure 4.46). A suction of 200 kPa has shifted the cone resistance plot only about 12% for a sand specimen subjected to a constant isotropic net confining stress of 150 kPa (Figure 4.47).

Comparing the results for both relative densities reveals that the effect of suction is more pronounced for lower relative densities. It is also evident that the contribution of suction to cone resistance becomes more significant as the confining stress decreases.
and thus the effect of suction is more important for shallow penetrations (up to 5m) where soil is most likely to be unsaturated and geostatic stresses are low.

**4.3.6 Post test core sampling**

Post test core samples, using a hand operated auger of 50 mm diameter, were taken from different locations of the chamber specimens to investigate the moisture distribution and variation in particle size distributions resulting from localized particle crushing (Figure 4.48).

The moisture content analysis indicates that the specimen formation method adopted leads to an even moisture distribution throughout a specimen (Appendix D). There is also a good agreement between post test measured moisture contents and those obtained from soil water characteristic curves for the corresponding values of applied suction.

To investigate particle crushing, particle size distribution analyses were performed on core samples obtained from middle of the specimen (where cone penetration occurred) and at the edges of the specimen (within a distance of about 30 mm from rubber membrane). Only core samples recovered between depths of 0.35 m and 0.55 m were used.

Figure 4.49 shows the particle size distributions for the test UM50-200 (having $D_r = 61\%$, isotropic net confining stress = 50 kPa, $s = 200$ kPa, $q_c = 6.7$ MPa). It is observed that the particle size distribution of the soil obtained from the edge of the specimen is identical to that before cone penetration testing. However, the percentage of fines in the soil obtained from the middle of the specimen after cone penetration testing increased indicating that some particle crushing occurred at that location. As the soil was taken from a 50 mm diameter core, that is within an annulus extending 17 mm from the edge of the penetrating cone (having a diameter of 16 mm), the observed particle size distribution represents an average throughout this annulus.

It is probable that a significant variation of the amount of particle crushing exists within the annulus. However, more advanced methods of sampling would be required
to confirm this, and more generally to quantify the extent of particle crushing at the cone tip and its variation with distance from the cone tip.

4.3.7 Less successful specimen formation procedures
Two other methods of forming unsaturated sand specimens were studied but were found to be less successful than the method described above. In one method a dry sand specimen was formed using the pluviation method described above and then de-aired water was added to achieve a moisture content 2% higher than the target moisture content (obtained from the soil–water characteristic curve for the target suction). The water was poured uniformly across the gravel layer at the top of the specimen. The chamber was then assembled and lateral and vertical pressures applied. The target suction was applied by increasing the pore air pressure at the top of the specimen and maintaining a constant water pressure at the base of the specimen. It was supposed that this procedure would result in a uniform distribution of moisture throughout the specimen; however, post equilibrium analysis of the specimen indicated that the procedure resulted in a preferential flow of water within the specimen leading to zones of high saturation separated by zones of relatively dry sand.

The other specimen preparation method studied involved moist tamping of equal layers of loosely deposited sand. The sand in each layer had a moisture content related to the target suction. This method resulted in a relatively uniform distribution of moisture within the specimen. However, CPT results indicated non-uniform densities within each layer due to the compaction procedure adopted. This is clearly evidenced by the sinusoidal cone penetration resistance response with depth as shown in Figure 4.50. Note that Figure 4.50 corresponds to moist tamping method.

4.4 CONCLUDING REMARKS
Dry sand specimens were prepared in the calibration chamber using the pluvial deposition technique. A targeted relative density was reached by adjusting the sand flow rate and drop height. Unsaturated specimens were formed by first saturating the specimens and then letting the moisture content reduce to achieve a target suction.
Suction was applied and controlled using axis translation technique. Cone penetration tests were conducted with the standard penetration rate using a miniature electrical cone.

Several CPTs were conducted on specimens subjected to a range of relative densities, confining net stresses and initial controlled suctions to investigate the CPT in unsaturated soils. The main observations are summarized as follows:

- Suction noticeably increases the average cone resistance in sands and the effect of suction is less prominent for small suction values.

- Suction can increase cone resistance by as much as 90% for a net confining stress of 25 kPa.

- Suction increases cone resistance by as much as 31% for a net confining stress of 100kPa.

- The contribution of suction to the cone resistance becomes more significant as the net confining stress decreases and thus the influence of suction is more important for shallow penetrations (up to 5 m) where soil is most likely to be unsaturated and geostatic stresses are low.

- The effect of suction on the cone resistance is more pronounced for loose specimens.

A very good repeatability of test results was demonstrated confirming the suitability of the specimen formation and testing procedures adopted. Post test core samples taken from different locations of the chamber specimens confirmed an even distribution of moisture content throughout the specimen. The core sample obtained from middle of the chamber specimen indicated that some particle crushing occurred in vicinity of the location that the cone penetrated. However, more advanced methods of sampling
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![Cone resistance, $q_c$ (MPa)](chart)
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5.1 INTRODUCTION

This chapter presents the constitutive model used to describe the stress-strain behavior of Sydney sand. It should be noted that constitutive modelling is not among the main original contributions of this thesis as outlined in chapter 1. The constitutive model used here is a simplified version of the Russell and Khalili (2004 and 2006a) bounding surface plasticity model. The simplification involved forcing the bounding and loading surfaces to coincide so that they represent a single yield surface. The simplification was made so that the model becomes of the conventional elastic plastic type which are more widely used by the geotechnical engineering community. The simplification required the model to be recalibrated as will be detailed in this chapter.

The model is calibrated for Sydney sand using the results of an extensive program of laboratory tests conducted by Russell (2004) as detailed in Appendix C. Model simulations of the triaxial compression test results for saturated and unsaturated sand are presented in this chapter. The calibrated model is implemented into an analysis of cavity expansion in soils of finite and infinite extent in chapter 6. The cavity expansion
analysis is used to evaluate the boundary effects on CPT results from calibration chambers in chapter 7.

5.2 NOTATION

Conventional triaxial $p' - q$ notation is used, where $p'$ is the mean effective stress and $q$ is the deviator stress. The corresponding work conjugate strain variables are the soil skeleton volumetric (isotropic) strain $\varepsilon_p$ and shear (deviatoric) strain $\varepsilon_q$. They are related to axial and radial stresses and strains in the usual way, where:

$$
p' = \frac{\sigma'_1 + 2\sigma'_3}{3}, \quad q = \sigma'_1 - \sigma'_3, \quad \varepsilon_p = \varepsilon_1 + 2\varepsilon_3, \quad \varepsilon_q = \frac{2}{3}(\varepsilon_1 - \varepsilon_3)
$$

and subscripts 1 and 3 denote the axial and radial components, respectively.

Compressive stresses and strains are assumed positive and volumetric strain is linked to specific volume ($\nu$) according to:

$$
\varepsilon_p = -\ln\left(\frac{\nu}{\nu_0}\right)
$$

where $\nu = 1 + e$, $e$ is the void ratio and $\nu_0$ is the specific volume at the reference configuration. In incremental form Equation (5.2) can be rewritten as:

$$
\dot{\varepsilon}_p = -\dot{\nu} \frac{\dot{\nu}}{\nu}
$$

where a superimposed dot indicates an increment. Elastic and plastic strain increments sum to give total strain increments in the usual way:

$$
\begin{bmatrix} \dot{\varepsilon}_p \\ \dot{\varepsilon}_q \end{bmatrix} = \begin{bmatrix} \dot{\varepsilon}_p^e \\ \dot{\varepsilon}_q^e \end{bmatrix} + \begin{bmatrix} \dot{\varepsilon}_p^p \\ \dot{\varepsilon}_q^p \end{bmatrix}
$$

(5.4)
where the superscripts $e$ and $p$ denote the elastic and plastic components, respectively. The pairs of stresses and strains may be abbreviated in vector form $\sigma = [p', q]'$ and $\varepsilon = [\varepsilon_p, \varepsilon_q]'$.

### 5.3 THE CONSTITUTIVE MODEL

The ingredients of a conventional elastic-plastic constitutive model for unsaturated sands formulated within the critical state framework are presented here. Compression is assumed positive and rate effects are ignored.

#### 5.3.1 Effective stress

The effective stress (Bishop, 1959) is defined as:

$$\sigma' = \sigma_n + \chi s$$  \hspace{1cm} (5.5)

where a superimposed dash denotes the stress invariant to be effective; $\sigma_n$ is the total stress in excess of pore air pressure ($u_a$); also referred to as net stress; $s$ is the difference between pore air pressure and pore water pressure ($u_a - u_w$) referred to as matric suction; and $\chi$ is the effective stress parameter. For saturated soils, $\chi = 1$, and Equation 5.5 reduces to the conventionally used definition $\sigma' = \sigma_n - u_w$.

Khalili and Khabbaz (1998) obtained a unique relationship for the effective stress parameter ($\chi$) in terms of the suction ratio $s/s_e$:

$$\chi = \begin{cases} 
1 & \text{for } \frac{s}{s_e} \leq 1 \\
\left(\frac{s}{s_e}\right)^{-0.55} & \text{for } \frac{s}{s_e} > 1 
\end{cases}$$ \hspace{1cm} (5.6)
where $s_e$ is the suction value representing transition between saturated and unsaturated states. For soils experiencing a reduction in degree of saturation along the main drying path, $s_e$ is equal to the air entry value ($s_{ae}$). For soils experiencing an increase in degree of saturation along the main wetting path, $s_e$ is equal to the air expulsion value ($s_{ex}$). Equation (5.6) was established using suction ratios less than 12. An extended relationship to include suction ratios greater than 25 was presented by Russell and Khalili (2006a):

$$
\chi = \begin{cases} 
1 & \text{for } \frac{s}{s_e} \leq 1 \\
\left(\frac{s}{s_e}\right)^{-0.55} & \text{for } 1 < \frac{s}{s_e} \leq 25 \\
25^{0.45} \left(\frac{s}{s_e}\right)^{-1} & \text{for } \frac{s}{s_e} > 25
\end{cases}
$$

(5.7)

5.3.2 The critical state

The critical state acts as a reference condition towards which all states approach with increasing plastic shear strain. In this study the critical state line (CSL) in the $v \sim \ln p'$ plane for saturated sands is assumed to take the form of three linear segments (illustrated in Figure 5.1) as identified by Russell and Khalili (2002) through a review of several sets of experimental data across a wide stress range.

The three linear segments are defined by the six material parameters $\lambda_0$, $\Gamma_0$, $v_{cr}$, $\lambda_{cr}$, $v_f$ and $\lambda_f$, where $\lambda_0$ and $\Gamma_0$ are the slope of the initial portion of the CSL and its specific volume at $p' = 1$ kPa, respectively; $v_{cr}$ is the specific volume at the onset of particle crushing; $\lambda_{cr}$ is the slope during the particle crushing stage; and $v_f$ and $\lambda_f$ are the specific volume at the end of crushing and the slope of the CSL at extremely high stresses, respectively. For saturated conditions a general definition of the CSL in the $v \sim \ln p'$ plane is used in the model formulation for simplicity:
\[ v = f_{cs}(p') \]  \hspace{1cm} (5.8)

where \( f_{cs} \) is a function unique to a given soil. Any function for \( f_{cs} \) can be chosen as long as it closely fits the three linear segments in Figure 5.1. Russell and Khalili (2002) suggested one possible function, which introduces slight curvatures in the CSL at the two points where the linear segments join.

In the \( q \sim p' \) plane the CSL is taken to be linear and pass through the origin. Its slope \( M_{cs} \) is a material constant, irrespective of the amount of particle crushing that the sand may have experienced, as indicated by experimental data (see for example Russell and Khalili, 2004).

Adopting the Mohr-Coulomb failure criterion it can be shown that \( M_{cs} \) is simply a function of the critical state friction angle \( (\phi'_{cs}) \) according to:

\[ M_{cs} = \frac{6 \sin \phi'_{cs}}{3t - \sin \phi'_{cs}} \]  \hspace{1cm} (5.9)

where \( M_{cs} \) and \( \phi'_{cs} \) are material constants, \( t = +1 \) for compressive loading \( (q > 0) \) and \( t = -1 \) for extensive loading \( (q < 0) \).

### 5.3.3 Elasticity

A simple isotropic elastic rule is adopted and is the same as that used for both sands and clays in many other constitutive models. Specifically, incremental elastic volumetric strain accompanies a change in \( p' \) according to a linear relationship between \( v \) and \( \ln p' \) such that the elastic bulk modulus, \( K \), is defined as:

\[ K = \frac{v p'}{\kappa} \]  \hspace{1cm} (5.10)
where $\kappa$ is a material constant and represents the slope of the elastic unload-reload line in the $v \sim \ln p'$ plane. For triaxial conditions the elastic shear modulus, $G$, is then defined as:

$$G = \frac{3(1-2\mu)}{2(1+\mu)}K$$  \hspace{1cm} (5.11)

where $\mu$ is Poisson’s ratio.

### 5.3.4 Elastic-plastic stress-strain relationships

Incremental elastic strains are linked to the incremental stress invariant through:

$$\dot{\varepsilon} = D^e \dot{\varepsilon}^e$$  \hspace{1cm} (5.12)

where $D^e$ is the elastic stiffness matrix defined as:

$$D^e = \begin{bmatrix} K & 0 \\ 0 & 3G \end{bmatrix}$$  \hspace{1cm} (5.13)

The plastic stress-strain relationship is of the form:

$$\dot{\varepsilon}^p = \frac{1}{h} (\mathbf{n}^T \dot{\mathbf{\sigma}}) \mathbf{m}$$  \hspace{1cm} (5.14)

where $\mathbf{n} = [n_p, n_q]^T$ is the unit normal vector at the current stress state on the yield surface controlling the direction of loading, $\mathbf{m} = [m_p, m_q]^T$ is the unit direction of plastic flow at the current stress state and $h$ is the hardening modulus.

The elastic-plastic stress-strain relationship is obtained by combining Equations (5.4), (5.12) and (5.14) and is of the form:
5.3.5 Yield surface and isotropic compression line

When using conventional plasticity theory, in the $q \sim p$ plane the purely elastic region is assumed to be enclosed by the yield surface. Plastic deformation occurs when the current stress state, $\sigma$ is located on the yield surface. The yield surface function adopted here is:

$$f = q - M_{cs} p \left[ \ln \left( \frac{p'_e}{p} \right) \right]^{1/N} = 0$$

(5.16)

where $p'_e$ is an isotropic hardening parameter controlling the size of the surface. It represents intercept of yield surface with the $q = 0$ axis as illustrated in Figure 5.2. The material constant $N$ controls the curvature of the surface. $R_m$ is another material constant that will be defined below.

Inherent in definition of this model is the existence of an isotropic compression line (ICL) in the $v \sim \ln p'$ plane, towards which the trajectories of all isotropic compression load paths approach. For saturated conditions the ICL is expressed as:

$$v = f_{cs} \left( \frac{p'}{R_m} \right) - \kappa \ln R_m$$

(5.17)

The material constant $R_m$ therefore represents the constant shift of the ICL along a $\kappa$ line from the CSL in the $v \sim \ln p'$ plane (Figure 5.3).

Both $N$ and $R_m$ can be determined from the undrained response of the material at its loosest state in the $q \sim p'$ plane.
The unit normal vector at \( \sigma \) defining the direction of loading is then:

\[
\mathbf{n} = \frac{\partial f}{\partial \sigma} = \frac{\left( \begin{array}{c} - \frac{q}{p'} \left( 1 - \frac{1}{N \ln(p'_c/p')} \right) \\ \left( - \frac{q}{p'} \left( 1 - \frac{1}{N \ln(p'_c/p')} \right) \right) + 1 \end{array} \right)}{\left( \begin{array}{c} 1 \\ \left( - \frac{q}{p'} \left( 1 - \frac{1}{N \ln(p'_c/p')} \right) \right)^2 + 1 \end{array} \right)}^T
\]  

(5.18)

5.3.6 Plastic potential

The plastic potential \((g = 0)\) controls the ratio between the incremental plastic volumetric strain and plastic shear strain. A non-associated flow rule is assumed that ensures the plastic volumetric strains are zero at the critical state. The dilatancy, \(d\), is expressed as:

\[
d = \frac{\dot{\varepsilon}_p^{\dot{p}}}{\dot{\varepsilon}_q} = (1 + k_d \zeta)M_{cs} - \frac{q}{p'}
\]  

(5.19)

where \(\zeta\) is the state parameter (Been and Jefferies, 1985) defined as the vertical distance between the current state and CSL in the \(v \sim \ln p'\) plane and \(k_d\) is a material parameter. The state parameter is positive when the current state is above the CSL, is negative when the current state is below the CSL and is equal to zero when current state is at the critical state.

The unit vector of plastic flow \((\mathbf{m})\) at \(\sigma\) is then defined by the general equation:

\[
\mathbf{m} = \frac{\partial g}{\partial \sigma} = \left[ \begin{array}{c} d \\ \frac{1}{\sqrt{1 + d^2}} \end{array} \right]^T
\]  

(5.20)

The function \(g = 0\), although not essential to operate the constitutive model, could be obtained by integrating Equation (5.19). For isotropic loading \(m_q\) is indefinite due to
non-smoothness of the plastic potential and singularity along \( q=0 \) axis. As such, for \( q=0 \) loading \( m_q \) is taken as 0.

### 5.3.7 Hardening rule

For saturated conditions the yield surface undergoes isotropic hardening with plastic volumetric strains in the usual way. By forcing \( \sigma \) to remain on the yield surface it follows that the hardening modulus \( h \) is defined as:

\[
h = -\frac{\partial f}{\partial p'_c} \frac{m_p}{\partial \sigma} \tag{5.21}
\]

where the first component on the right-hand side is obtained from Equation (5.16), the second component is obtained from Equation (5.17), and the remaining components are previously defined in Equations (5.18) and (5.20). In expanded form Equation (5.21) is expressed as:

\[
h = \frac{M_{c,v} p' \left[ \ln(p'_c / p') \right]^{1/N}}{N p'_c \ln(p'_c / p')} \cdot \frac{1}{\Pi} \cdot \frac{d}{\sqrt[4]{1 + d^2}}
\]

\[
\Pi = \frac{\partial p'_c}{\partial \varepsilon'_p} = \frac{v p'_c}{\lambda^* - \kappa} > 0
\]

\[
\lambda^* = -p'_c \frac{\partial v}{\partial p'_c}
\]
For unsaturated conditions, it is permissible that \( p'_c \) undergoes isotropic hardening with changes in suction as well as plastic volumetric strains, similar to the assumptions of Loret and Khalili (2000) and Loret and Khalili (2002). For collapsible soils, suction hardening occurs as a result of \( p'_c \) increasing at a faster rate than \( p' \) during an increase in \( s \) (Khalili et al., 2004). It follows that:

\[
h = -\frac{\partial f}{\partial p'_c} \left( \frac{\partial p'_c}{\partial \varepsilon^p_p} + \frac{\partial p'_c}{\partial s} \frac{\dot{\varepsilon}^p_p}{\dot{\varepsilon}^p_p} \right) \frac{m_p}{\left\| \sigma \right\|} \tag{5.25}
\]

More specifically the ICL may undergo a suction dependant shift \( \gamma(s) \) along the \( \kappa \) line in the \( v \sim \ln p' \) plane as illustrated in Figure 5.3. \( \gamma(s) \) is positive, has units of stress, and is subject to \( \gamma(s) = 0 \) when \( s \leq s_c \). Recalling that the saturated ICL is defined by Equation (5.17), an expression for the unsaturated ICL is of the form:

\[
v = f_{cs} \left( \frac{p' - \gamma(s)}{R_m} \right) - \kappa \ln \left( \frac{R_m p'}{p' - \gamma(s)} \right) \tag{5.26}
\]

Any shift of the unsaturated ICL is accompanied by a shift of the unsaturated CSL, assuming \( R_m \) is a material constant, and the unsaturated CSL is expressed as:

\[
v = f_{cs}(p', s) = f_{cs} \left( p' - \tilde{\gamma}(s) \right) - \kappa \ln \left( \frac{p'}{p' - \tilde{\gamma}(s)/R_m} \right) \tag{5.27}
\]

Using the procedure outlined by Loret and Khalili (2000) and Loret and Khalili (2002) it can be shown that the expression linking the plastic volumetric change (\( \Delta \nu^p \)) that occurs as the hardening parameter moves from the saturated ICL (\( p'_{c1} \)) to the unsaturated ICL (\( p'_{c2} \)) is:
\[ \Delta v^p = -(\lambda^* - \kappa) \ln \left( \frac{p_{c2}' - \gamma(s)}{p_{c1}'} \right) \]  
(5.28a)

\[ p_{c2}' = p_{c1}' \exp \left[ \frac{-\Delta v^p}{\lambda^* - \kappa} \right] + \gamma(s) \]  
(5.28b)

where \( \lambda^* \) is the slope of a straight line in the \( v \sim \ln p' \) plane that connects the points \( p_{c1}' \) and \( p_{c2}' - \gamma(s) \) on the saturated ICL. In the limit, Equation (5.28) becomes:

\[ \dot{p}' = \frac{\lambda^* - \kappa}{v} \left( \frac{\dot{p}_c' - \dot{\gamma}(s)}{p_c' - \gamma(s)} \right) \]  
(5.29a)

\[ \dot{p}_c' = \frac{v(p_c' - \gamma(s))}{\lambda^* - \kappa} \dot{\rho}_p^p + \left( \frac{\partial \gamma(s)}{\partial s} \right) \dot{s} \]  
(5.29b)

where \( \lambda^* \) is the slope of the saturated ICL in the \( v \sim \ln p' \) plane at \( p_c' - \gamma(s) \). \( \lambda^* \) is therefore independent of \( \gamma(s) \).

### 5.3.8 Coupling the solid, air and water phases

If \( s \) is constant during soil deformation, the volume of water in the sample will vary, as will the volume of air. Conversely, \( s \) will vary during deformation if either the air or water volumes are held constant. It is therefore necessary to couple \( s \) with these volumes. The procedure for coupling the three phases of an unsaturated material in an effective stress framework is detailed by Khalili et al. (2000) and only the basic features are presented here. Note that \( v_w = s, e \) is a measure of pore water volume and \( v_a = (1 - s, e) \) is a measure of pore air volume.

By adopting Betti’s reciprocal rule it can be shown that for a material with incompressible grains (Khalili et al., 2000):
where $c$ is the drained compressibility of the soil skeleton, $c_m$ represents the compressibility of the soil skeleton with respect to $s$, $c_m'$ is the compressibility of the water phase with respect to $s$ and $\psi$ is the incremental effective stress parameter ($\psi = \frac{c_m}{c}$).

It can be shown that:

\[
\begin{align*}
\dot{c} &= -\left(\frac{\partial v}{\partial v}\right)\frac{\partial p_n}{\partial s} & \text{when } \dot{s} = 0 \\
\dot{c}_m &= -\left(\frac{\partial v}{\partial v}\right)\frac{\partial s}{\partial s} & \text{when } \dot{p}_n = 0 \\
\dot{c}_m' &= -\left(\frac{\partial v_u}{\partial v}\right)\frac{\partial s}{\partial s} & \text{when } \dot{p}_n = 0
\end{align*}
\]

in which $p_n$ is the mean net stress.

The compressibility coefficient, $c$, is equal to the inverse of the drained bulk modulus of soil skeleton ($c = \frac{1}{K}$).

The form of $c_m'$ is dependant on the assumed soil-water characteristic curve (SWCC). For example if the SWCC has a constant slope of $\frac{\partial w}{\partial s}$ in the $w \sim s$ plane for all values of $s > s_e$ (as presented in Appendix C) then an alternative form of Equation (5.34) may be derived (Russell, 2004):
\[ e'_m = \left( \frac{\psi - S_r}{v} \right) \frac{\dot{v}}{S} + \psi^2 c - \left( \frac{v - 1}{v} \right) \frac{\partial S_r}{\partial S} \]  

(5.35)

The incremental effective stress parameter (\( \psi \)) is derived by differentiating \( \chi_s \) with respect to \( s' \):

\[ \psi = \frac{\partial (\chi_s)}{\partial s} = \begin{cases} 
1 & \text{for } \frac{s}{s_c} \leq 1 \\
0.45 \left( \frac{s}{s_c} \right)^{-0.55} & \text{for } 1 < \frac{s}{s_c} \leq 25 \\
0 & \text{for } \frac{s}{s_c} > 25 
\end{cases} \]  

(5.36)

5.4 MODEL CALIBRATION

This section describes the calibration of the constitutive model for Sydney sand using the experimental results of Russell (2004) presented in Appendix C.

Results of undrained triaxial tests conducted on the loosest specimens are used to determine initial estimates of the parameters \( N \) and \( R_m \) which define the shape of the yield surface in the \( q \sim p' \) plane and the shift of the ICL from CSL in the \( v \sim \ln p' \) plane (Figure 5.3). Firstly it was assumed that \( k \) is very small and initial estimates of \( N \) and \( R_m \) were obtained by fitting the yield surface to the undrained triaxial test results conducted on the loosest samples and plotted in the \( q/p'_0/M_{cs} \sim p'/p'_0 \) plane, where the subscript 0 denotes the condition at the start of the test. For extremely loose samples the contribution of elasticity to strains is negligible and the initial state of the test is very close to the ICL and therefore test results follow the yield surface closely. For determination of the parameter \( R_m \) the isotropic compression line has also been considered.

The plastic potential (Equation (5.19)) was determined by analysis of incremental strains of saturated drained triaxial test results. Generally, drained triaxial test results
are not sensitive to elastic straining and the contribution of elasticity may be neglected (e.g. Jefferies, 1993; Muir Wood et al., 1994). Therefore, in the analysis of incremental strains, plastic strain increments were assumed to be equal to the total strain increments. Specific values of $k_d$ was found to suit a given range of specific volumes.

The next step in the calibration of the model was to introduce elastic strains. A typical value of $\mu = 0.3$ was assumed and a suitable value for $\kappa$ was determined using a trial and error procedure to fit the model simulations with the experimental results presented in Appendix C. After introducing the elastic strains defined by the elastic parameters the calibration procedure was repeated to identify if any parameter was required to be redefined.

5.5 MODEL SIMULATIONS

This section compares model simulations with selected experimental results of Russell (2004), presented in full in Appendix C. Firstly material parameters determined for Sydney sand are summarized. Then the saturated and unsaturated triaxial compression shear test results along with corresponding simulations are presented.

5.5.1 Input parameters

The constitutive model was calibrated for Sydney sand using the procedure described above in order to simulate a series of drained and undrained triaxial compression tests reported by Russell (2004).

The parameters $\lambda_0 = 0.0284$, $\Gamma_0 = 2.0373$, $\nu_{cr} = 1.835$, $\lambda_{cr} = 0.195$, $\nu_f = 1.25$ and $\lambda_f = 0.04$ define the three segmented CSL in the $\nu \sim \ln p'$ plane. Also, $\phi'_{ts} = 36.3^0$ was found to fit the data well for saturated conditions irrespective of stress level. The elastic parameters were found to be $\kappa = 0.006$ and $\mu = 0.3$ and values of $N = 3$ and $R_m = 7.3$ were found to be appropriate. The material parameters $k_d = 2.8$, 1.9 and 1.0 were used for specific volumes $\nu = 1.823$, 1.726, 1.629, corresponding to $D_r = 30\%$, 60\% and 90\%, respectively.
The air entry value \((s_e)\) was found to be equal to 7 kPa for a range of void ratios. Also, the SWCC for Sydney sand was defined as (after Russell, 2004):

\[
S_r = \begin{cases} 
1 & \text{for } s \leq s_e \\
\left(\frac{s}{s_e}\right)^{-7.8} & \text{for } s_e < s \leq s_r \\
0.0274G_s\left(\frac{s}{s_e}\right)^{-0.23} & \text{for } s > s_r
\end{cases}
\] (5.37)

where \(s_r\) is residual suction value. Differentiation of \(S_r\) with respect to \(s\) gives:

\[
\frac{\partial S_r}{\partial s} = \begin{cases} 
0 & \text{for } s \leq s_e \\
-7.8\left(\frac{s}{s_e}\right)^{-8.8} & \text{for } s_e < s \leq s_r \\
-0.0274G_s\frac{0.23}{s_e}\left(\frac{s}{s_e}\right)^{-1.23} & \text{for } s > s_r
\end{cases}
\] (5.38)

As pointed out by Russell and Khalili (2006a) the CSL is unique for saturated and unsaturated Sydney sand. No suction hardening occurs for this material and therefore:

\[
\frac{\partial \sigma^t}{\partial s} = 0 \quad \text{and} \quad \gamma(s) = 0
\] (5.39)

### 5.5.2 Simulation of triaxial test results

Figures 5.4 and 5.5 present the saturated drained triaxial test results and model simulations in the \(\sigma_1 - \epsilon_q\) and \(\sigma_3 - \epsilon_q\) planes. The model simulations are shown with the continuous lines and a solid symbol, whereas the test results are represented by a hollow symbol of the same shape. The model simulations for drained tests on dense samples exhibit the classical stress-strain behavior. Specifically, hardening occurs up
to a peak in the shear resistance, accompanied by initial volumetric contraction followed by noticeable volumetric dilation. After reaching the peak, softening towards the critical state line is observed.

Figure 5.6 shows saturated undrained triaxial test results and model simulations in the $q \sim p'$ and $q \sim \varepsilon_q$ planes. The continuous and dashed lines represent model simulations and hollow symbols are test results. The classical behavior of the saturated undrained tests conducted on loose and dense samples is simulated by the model. Specifically, the sample initially looser than critical exhibits a peak in shear resistance followed by a gradual reduction. For the dense sample the shear resistance increases continuously from the start of compression.

Figures 5.7 and 5.8 present the unsaturated constant suction triaxial test results and model simulations in the $q \sim \varepsilon_q$ and $\varepsilon_p \sim \varepsilon_q$ planes. The model simulations are shown with the continuous lines and a solid symbol, whereas the test results are represented by a hollow symbol of the same shape. There is a reasonable agreement between simulation and experiment, especially at large strains. Also the stress-strain behavior is analogous to that of saturated drained sands initially denser than critical. Specifically, hardening occurs up to a peak in the shear resistance, accompanied by initial volumetric contraction. Softening towards the CSL is observed after reaching the peak and is accompanied by volumetric dilation.

The poor fit between simulation and experiment at small strains is a symptom of the conventional elastic-plastic theory used in the model formulation. More advanced models based on more complicated plasticity theories, such as bounding surface plasticity theory, are needed to capture the highly non-linear material response in the small strain region and the gradual mobilization of peak strength. However, the preference here is to use a model based on conventional elastic-plastic theory, as they are more widely used by the geotechnical engineering community, even though they provide less accurate simulations of the stress-strain response. An improved fit could be achieved with very different expressions that define highly non-linear elasticity (like those in Shuttle & Jefferies, 1998, for example) but at the expense of added complexity. Here the preference is to adopt simple elasticity assumptions that are well known through Cam-clay type critical state models.
There is a good fit between simulation and experiment at large shear strains, especially as the critical state is approached. This is an important feature as cavity expansion causes a soil to undergo large strains and, as will become clear in Chapter 6, the critical state is reached by the soil at the cavity wall.

5.6 CONCLUDING REMARKS
A simplified constitutive model in the critical state and conventional plasticity framework has been presented that is suited to describe the stress strain behavior of saturated and unsaturated sands. The concept of effective stress in unsaturated soils has been incorporated in the definition of the basic model ingredients. Also a simple isotropic elastic rule was adopted along with a non-associative flow rule. The model takes into account hardening due to plastic volumetric strains as well as suction in unsaturated conditions. The model was calibrated for saturated and unsaturated Sydney sand to provide a good match between simulation and triaxial tests results for saturated and unsaturated conditions. The match is particularly good at large shear strains as the critical state is approached. Implementation of the model into a cavity expansion analysis is presented in chapters 6 and 7.
Figure 5.1 A general critical state line for saturated sands in the $v \sim \ln p'$ plane.

Figure 5.2 Yield surface, hardening parameter and critical state line (CSL) in the $q \sim p$ plane.
Figure 5.3 Saturated and unsaturated CSLs and ICLs in the $\nu \sim \ln p'$ plane.
Figure 5.4 Saturated drained triaxial test results and model simulations for tests 50D, 242D presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant value of $\sigma'_3 = 50$ kPa, 242 kPa and initial specific volume of $v_0 = 1.677, 1.735$ respectively.
Figure 5.5 Saturated drained triaxial test results and model simulations for tests 115D, 301D presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant value of $\sigma'_3 = 115$ kPa, 301 kPa and initial specific volume of $v_0 = 1.685$, 1.730 respectively.
Figure 5.6 Saturated undrained triaxial test results and model simulations for tests 300U-L, 300U-D presented in the $q \sim p'$ (a) and $q \sim \varepsilon_q$ (b) planes, with constant value of $\sigma'_3 = 300$ kPa and initial specific volume of $v_0 = 1.917, 1.721$ respectively.
Figure 5.7 Unsaturated constant suction triaxial test results and model simulations for tests 5050L-D and 10050D-D presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant value of $\sigma'_3 = 50 \text{kPa}$, 102 kPa and initial specific volume of $v_0 = 1.77, 1.658$ respectively, each with a constant $s$ value of 51 kPa.
Figure 5.8 Unsaturated constant suction triaxial test results and model simulations for tests 50200L-D and 100200D-D presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant value of $\sigma'_3 = 51$ kPa, 100.5 kPa, initial specific volume of $v_0 = 1.780, 1.697$ and $s = 198$ kPa, 199.5 kPa respectively.
CHAPTER 6

DRAINED CAVITY EXPANSIONS IN SOILS OF FINITE RADIAL EXTENT

6.1 INTRODUCTION

In this chapter a new analytical procedure for solving the drained cavity expansion problem in soils of finite radial extent is presented (Pournaghiazar et al., 2011b). The procedure is novel as it enables conventional elastic-plastic hardening/softening constitutive models formulated in a critical state framework to be used in the analysis, as previously it has been possible to use only the elastic perfectly plastic Mohr-Coulomb model in an analysis of this type.

Both cylindrical and spherical cavities are considered in the analysis, and another novelty is that results are generated, presented and compared for both constant stress and zero displacement boundary conditions for the first time. Results are generated for Sydney sand using the conventional elastic-plastic constitutive model presented in chapter 5. The results highlight that both the boundary extent and type significantly influence the pressures required to expand cylindrical and spherical cavities.
In chapter 7 the analysis will be used to explain calibration chamber size effects reported in the literature and to present a method for correcting cone penetration resistances measured in a calibration chamber to what would be observed in free field conditions.

The drained cavity expansion analysis presented here is equally applicable to saturated (constant pore water pressure) and unsaturated (constant suction) conditions, as suction hardening is not observed in Sydney sand. A discussion is given on why the results may also be applicable to unsaturated undrained (constant moisture content) conditions, acknowledging the negligible differences between unsaturated drained and unsaturated undrained stress-strain behaviors in sands of the type considered here.

6.2 PRELIMINARIES AND NOTATIONS

The expanding cavity is surrounded by soil in which deformation comprises a combination of elastic and plastic strains. This, in turn, is surrounded by soil in which deformation is purely elastic. The configuration is illustrated in Figure 6.1, where the radius of the expanding cavity, the radius of the elastic-plastic boundary and the radius of the finite boundary are denoted $c$, $R$ and $B$, respectively. The radius of a particular soil element is denoted using $r$. The soil’s stress-strain behavior is described using the conventional elastic-plastic constitutive model presented in chapter 5. Only cavities expanded from zero initial radius are considered.

Cylindrical polar notations ($r, z, \theta$) are used for cylindrical cavities and spherical polar notations ($r, \phi, \theta$) are used for spherical cavities. Due to the kinematic constraints the radial stress ($\sigma_r'$) represents the major principle stress and the tangential (hoop) stress ($\sigma_\theta'$) represents the minor principle stress (Carter et al., 1986) as indicated in Figure 6.1. For cylindrical cavities $\sigma_\phi'$ represents an intermediate stress and for spherical cavities, due to symmetry, the intermediate stress ($\sigma_\phi'$) is equal to $\sigma_\theta'$. The mean effective stress $p'$ and deviatoric stress $q$ are defined in terms of $\sigma_r'$ and $\sigma_\theta'$ according to:
where \( k = 1 \) for the plane strain conditions of a cylindrical cavity and \( k = 2 \) for the radially symmetric conditions of a spherical cavity. The volumetric strain \( \varepsilon'_p \) and shear strain \( \varepsilon'_q \) are related to radial strain \( \varepsilon_r \) and tangential strain \( \varepsilon_\theta \) according to:

\[
\varepsilon'_p = \varepsilon_r + k\varepsilon_\theta \quad \varepsilon'_q = \frac{k}{k+1}(\varepsilon_r - \varepsilon_\theta)
\]  

(6.2)

where \( \varepsilon_r = 0 \) for cylindrical cavities and \( \varepsilon_\theta = \varepsilon_\phi \) for spherical cavities. The strains in Equation (6.2) have been chosen to ensure they are work conjugate to the stresses (Russell and Khalili, 2006b), that is \( \dot{p}' \dot{\varepsilon}_p + \dot{q} \dot{\varepsilon}_q = \dot{\sigma}'_r \dot{\varepsilon}_r + k \dot{\sigma}'_\theta \dot{\varepsilon}_\theta \).

Following Collins and Wang Yan (1990), it is further assumed that at the initial condition, when time \( t = 0 \), the cavity radius is \( c_0 = 0 \) and the soil element of interest is located at radius \( r_0 \). As time is increased, the cavity radius is assumed to be \( c \) and radius to the material element is \( r \) (Figure 6.1). The radial and tangential strains at \( r \) are defined in terms of radial displacement \( u \) according to:

\[
\varepsilon_r = -\frac{\partial u}{\partial r} \quad \varepsilon_\theta = -\frac{u}{r}
\]  

(6.3)

where \( u = r - r_0 \)

\( x \) is now used to denote any one of the stress or strain variables. \( \dot{\cdot} \) denotes the material time derivative of \( x \) at \( r \). \( \ddot{\cdot} \) denotes the local time derivative of \( x \) at \( r \). These are related by the expression (Collins et al., 1992):

\[
\ddot{x} = \dot{x} + \dot{w} \frac{\partial x}{\partial r}
\]  

(6.4)

where \( w \) is the radial velocity of the soil element at \( r \).
The radial and tangential strain rates at any instant are then defined in terms of \( w \) and \( r \) as:

\[
\varepsilon_r = -\frac{\partial w}{\partial r}, \quad \varepsilon_{\theta} = -\frac{w}{r}
\]  

(6.5)

It follows that:

\[
\varepsilon_r = -\frac{\partial w}{\partial r} \frac{kw}{r}, \quad \varepsilon_q = \frac{k}{k+1} \left( -\frac{\partial w}{\partial r} + \frac{w}{r} \right)
\]  

(6.6)

To ensure that a single set of elastic parameters \( E \) (Young’s modulus) and \( \mu \) (Poisson’s ratio) apply to both cylindrical and spherical cavities, the elastic bulk modulus \( K \) and shear modulus \( G \) are defined as:

\[
K = \frac{\rho'}{\varepsilon_p'} = \frac{\nu p'}{\kappa} = \frac{E}{(k+1)(1-2\mu)(1+(2-k)\mu)}
\]  

(6.7)

\[
G = \frac{k}{2(k+1)} = \frac{E}{2(1+(k-1)\mu)(1+(2-k)\mu)}
\]  

(6.8)

Using these notations \( \kappa \) would have to be slightly different for cylindrical and spherical cavity expansions. However, for all practical purposes, and for Poisson’s ratios typical of real soils, \( \kappa \) may be taken as a constant.

It is also necessary to rewrite the elastic stiffness matrix in Equation (5.13) so it is applicable to both cylindrical and spherical cavity expansions. The elastic stress-strain relationship then becomes (Collins and Stimpson, 1994):

\[
\begin{bmatrix}
\varepsilon_r' \\
p' \\
\varepsilon_q'
\end{bmatrix} =
\begin{bmatrix}
K & 0 \\
0 & \frac{2(k+1)}{k} G
\end{bmatrix}
\begin{bmatrix}
\varepsilon_r' \\
\varepsilon_q'
\end{bmatrix}
\]  

(6.9)
A definition for the slope of the CSL in the $q - p'$ plane, $M_{cs}$, applicable to both cylindrical and spherical cavity expansions is (Russell and Khalili, 2006b):

$$M_{cs} = \frac{2(k+1)\sin \phi'_{cs}}{(k+1)-(k-1)\sin \phi'_{cs}}$$

(6.10)

It is assumed here that if $\phi'_{cs}$ is the triaxial critical state friction angle then $\phi'_{cs} = \phi'_{tx}$ for spherical cavities ($k = 2$) and $\phi'_{cs} = 1.125\phi'_{tx}$ for cylindrical cavities ($k = 1$). This assumption (approximately) accounts for the effect of the intermediate principle stress on soil stress-strain behaviour during plane strain conditions that is otherwise ignored in the Mohr-Coulomb criterion (Wroth, 1984).

### 6.3 GOVERNING EQUATIONS

#### 6.3.1 Finite elastic region

In deriving the governing equations for the elastic region it is convenient to temporarily return back to polar notations. The stress-strain relations within the elastic region are:

$$\varepsilon_r = \frac{1+(2-k)\mu}{E} \left[ (1-2k)\mu \Delta \sigma_r - k\mu \Delta \sigma_\theta \right]$$

(6.11a)

$$\varepsilon_\theta = \frac{1+(2-k)\mu}{E} \left[ (1-\mu)\Delta \sigma_\theta - \mu \Delta \sigma_r \right]$$

(6.11b)

in which $\Delta \sigma_r = \sigma_r - p'_0$, $\Delta \sigma_\theta = \sigma_\theta - p'_0$ and $p'_0$ is the hydrostatic pressure applied to the soil medium before cavity expansion occurs.

The stress equilibrium can be written in a generalized way:

$$\frac{d\sigma_r}{dr} + k \frac{\sigma_r - \sigma_\theta}{r} = 0$$

(6.12)
The stress definitions, found by integration of the above, then take the general form (Salgado, 1993; Yu, 2000):

\[
\sigma_r = C_1 + k \frac{C_2}{r^{k+1}} \quad \sigma_\theta = C_1 - \frac{C_2}{r^{k+1}}
\]  

(6.13)

where the constants \( C_1 \) and \( C_2 \) are evaluated by invoking boundary conditions.

For the constant stress boundary condition the radial stress at the boundary of radius \( B \) is equal to \( p'_0 \) before cavity expansion as well as during cavity expansion. By denoting the radial stress at the elastic-plastic boundary of radius \( R \) using \( \sigma'_R \), it follows that stresses at any radius \( r \) throughout the elastic region (that is at radii between \( R \) and \( B \)) are given by (Salgado, 1993):

\[
\sigma'_r = p'_0 + (\sigma'_R - p'_0) \left[ \frac{\left(\frac{R}{r}\right)^{k+1} - \left(\frac{R}{B}\right)^{k+1}}{1 - \left(\frac{R}{B}\right)^{k+1}} \right] \quad (6.14a)
\]

\[
\sigma'_\theta = p'_0 - (\sigma'_R - p'_0) \left[ \frac{1 - \left(\frac{R}{r}\right)^{k+1} + \left(\frac{R}{B}\right)^{k+1}}{1 - \left(\frac{R}{B}\right)^{k+1}} \right] \quad (6.14b)
\]

For the zero displacement boundary condition and before cavity expansion the radial stress at the boundary of radius \( B \) is \( p'_0 \). Then, by ensuring \( \varepsilon_\theta = 0 \) at that boundary during expansion, the stresses at any radius \( r \) throughout the elastic region are:
The radial displacement $u$ can then be determined by substituting the appropriate stress definitions in to Equation (6.11b), and for constant stress boundary condition is:

$$u = \frac{\sigma'_r - p'_0}{1/k^{k+1} - 1/B^{k+1}} \frac{1 + (2-k)\mu}{E} \left[ \frac{1 + \mu(k-1)}{kr^k} + \frac{1 - 2\mu}{B^{k+1}} r \right]$$

(6.16)

and for the zero displacement boundary condition is:

$$u = -\frac{1 + (2-k)\mu}{Ek} \frac{r(\sigma'_r - p'_0)}{1 + \mu(k-1)} \left[ \frac{\mu(1-k)}{kr^k} + (1 + \mu(k-1)) \left( \frac{R^k}{r} \right)^{k+1} \right]$$

(6.17)

6.3.2 The elastic-plastic region

Five differential equations govern drained (constant pore water pressure or suction) cavity expansions in the elastic-plastic region when the stress-strain behaviour is described using a conventional elastic-plastic constitutive model. They are:

- an equilibrium equation for stresses around the cavity;
- two constitutive equations;
- a consistency equation obtained by differentiation of the yield function with respect to time; and
- a continuity equation for the solid mass.
Returning to \( p' : q \) notations, the equilibrium equation becomes:

\[
\frac{\partial p'}{\partial r} + \frac{k}{k+1} \frac{\partial q}{\partial r} + \frac{kq}{r} = 0
\]

(6.18)

One constitutive equation is written by linking the volumetric strain components to incremental stresses:

\[
\varepsilon_p^e - \varepsilon_p^v = \frac{1}{h} \left( n_p p' + n_q q \right) m_p
\]

(6.19)

in which Equations (6.6) and (6.7) can be substituted to give:

\[
\frac{\partial w}{\partial r} + \frac{kw}{r} \frac{p'}{K} + \frac{kq}{r} \left( n_p p' + n_q q \right) m_p = 0
\]

(6.20)

Similarly, using the deviatoric components of strain, the second constitutive equation is:

\[
\frac{k}{k+1} \left( \frac{\partial w}{\partial r} - \frac{w}{r} \right) + \frac{kq}{2(k+1)G} + \frac{1}{h} \left( n_p p' + n_q q \right) m_q = 0
\]

(6.21)

The yield function is expressed in rate form to obtain the consistency equation:

\[
\frac{\partial f}{\partial p'} + \frac{\partial f}{\partial q} + \frac{\partial f}{\partial p_c} = 0
\]

(6.22)

The continuity equation linking the rate of volumetric change of the soil skeleton to the total strain rates ensuring conservation of solid mass is:

\[
\frac{\partial w}{\partial r} + \frac{kw}{r}
\]

(6.23)
6.4 SOLUTION PROCEDURE FOR CAVITY EXPANSION IN FINITE MEDIA

In the elastic region, the solution procedure is straightforward and involves using the closed form expressions for stresses, strains and displacement (or velocity) in terms of \( r, R \) and \( B \).

In the elastic-plastic region the solution procedure is far more complicated and involves converting Equations (6.18), (6.20), (6.21), (6.22) and (6.23) to a system of differential equations. In this case, all stress and strain variables surrounding an expanding cavity are functions of \( r, R \) and \( B \), which can be grouped into two non-dimensional radii as:

\[
\eta = \frac{r}{R} \text{ and } \xi = \frac{B}{R} \tag{6.24}
\]

although other combinations of \( r, R \) and \( B \) could be used in the groupings. Again using \( x \) to denote any of the variables it follows that:

\[
x = f(\eta, \xi) \tag{6.25}
\]

Using \( W \) to denote the velocity of expansion of the elastic–plastic boundary it follows that:

\[
\eta = \frac{r}{W_t} \text{ and } \xi = \frac{B}{W_t} \tag{6.26}
\]

The local space derivative of \( x \) at \( r \) is:

\[
\frac{\partial x}{\partial r} = \frac{1}{R} \frac{dx}{d\eta} \tag{6.27}
\]

This is independent of \( \xi \) as \( \xi \) is independent of \( r \).

The local time derivative of \( x \) at \( r \) is:
\[
\frac{dx}{dt} = \frac{d\eta}{dt} \frac{dx}{d\eta} + \frac{d\xi}{dt} \frac{dx}{d\xi} = \frac{\eta}{t} \frac{dx}{d\eta} - \frac{\xi}{t} \frac{dx}{d\xi}
\]  

(6.28)

Therefore, according to Equation (6.4) the incremental form of \(x\) can be converted to differential form using:

\[
x = \frac{W}{R} \left( (\tilde{w} - \eta) \frac{dx}{d\eta} - \xi \frac{dx}{d\xi} \right)
\]  

(6.29)

where \(\tilde{w} = \frac{w}{W}\) is a non-dimensional velocity at \(r\).

Equations (6.27) and (6.29) enable differential equations to be created in terms of \(\eta\) and \(\xi\). However, as the differential equations are expressed in terms of two variables, their analytical solution using a conventional elastic-plastic hardening/softening model of the type considered here is a formidable task.

However, the solution procedure may be simplified by introducing a small approximation. Attention is given to the limiting condition of cavity expansion for which \(\xi = \frac{B}{R}\) may be taken as a shape factor and constant. For this condition the \(\xi\) part of Equation (6.28) may then be assumed zero, and the differential equations are defined in terms of \(\eta\) only. The errors associated with this approximation which depend primarily on the value of \(\xi\), are less than 10% for input parameters relevant to real soils and values of \(\xi\) encountered in calibration chambers (see section 6.5). It will be demonstrated in section 6.5 that the solution procedure becomes much simpler when using the elastic perfectly plastic Mohr-Coulomb model, as throughout the elastic-plastic region the stress ratio \(q/p'\) is constant which removes a stress variable’s dependence on strain.

It is important to note (as will become evident below) that even when this simplification of constant \(\xi\) is applied, the cavity expansion problem still depends on \(\xi\) and the finite size of \(B\) and the constraints of the different boundary conditions at \(B\),
so is different to a cavity expansion problem formulated with the starting assumption of an infinite boundary.

In the elastic-plastic region the differential equations in terms of $\eta$ only are now presented. The solution procedure involves solving these simultaneously as an initial value problem to evaluate the stress and strains around the expanded cavity. The differential equations are:

$$
\begin{bmatrix}
C_{11} & C_{12} & 0 & 0 & 0 \\
C_{21} & C_{22} & 0 & C_{24} & 0 \\
C_{31} & C_{32} & 0 & C_{34} & 0 \\
C_{41} & C_{42} & 0 & 0 & C_{45} \\
0 & 0 & C_{53} & C_{54} & 0
\end{bmatrix}
\begin{bmatrix}
dp' / d\eta \\
dq / d\eta \\
dv / d\eta \\
dw / d\eta \\
dp_c' / d\eta
\end{bmatrix}
= 
\begin{bmatrix}
- k\bar{q} / \eta \\
- k\bar{q} / \eta \\
k\bar{w} / (k + 1)\eta \\
k\bar{w} / \eta \\
k\bar{w} / \eta
\end{bmatrix}
$$

(6.30)

where

$$
C_{11} = 1, \quad C_{12} = \frac{k}{k + 1},
$$

$$
C_{21} = \left(\bar{w} - \eta\right)\left(1 + \frac{n_pm_p}{h}\right)p', \quad C_{22} = \left(\bar{w} - \eta\right)\left(\frac{n_pm_p}{h}\right)p', \quad C_{24} = 1
$$

$$
C_{31} = \left(\bar{w} - \eta\right)\left(\frac{n_pm_q}{h}\right)p_r, \quad C_{32} = \left(\bar{w} - \eta\right)\left(\frac{k}{2(k+1)G} + \frac{n_pm_q}{h}\right)p_r, \quad C_{34} = \frac{k}{k + 1}
$$

$$
C_{41} = \frac{\partial f}{\partial p'}, \quad C_{42} = \frac{\partial f}{\partial q}, \quad C_{45} = \frac{\partial f}{\partial p_c'}
$$

$$
C_{53} = \frac{\bar{w} - \eta}{v}, \quad C_{54} = -1
$$

The first of the five rows of the matrix in Equation (6.30) corresponds to the equation of stress equilibrium, the next two to the constitutive laws and the remaining two to the consistency condition and continuity equation. The stress variables are put into non-dimensional form using a reference stress, $p'_r$, which is assumed to be 1kPa. The non-dimensional variables are denoted by a superimposed $\sim$. 
The stress conditions, specific volume and radial velocity at the elastic-plastic boundary are used as the initial values to solve Equation (6.30). A differential equation solver is then used to evaluate $p', q, v, w$ and $p'_c$ at any particular value of $\eta$ throughout the plastic region. Of particular interest is when $\eta = \bar{w}$ as it represents the soil state at the cavity wall. The stresses at this location also represent the cavity limit pressure which can be related to the cone resistance of a CPT. It is interesting to note that $\eta = \bar{w}$ corresponds to the condition of zero total volumetric strain rate (critical state for drained conditions) as controlled by the fifth row of the matrix in Equation (6.30).

If the analysis is to be applied to a calibration chamber study, where the radius of the cavity $c$ and the radial extent of the soil $B$ are analogous to the radius of the cone penetrometer and the radius of the specimen in the chamber, respectively, then an iterative procedure must be applied as follows:

1. Suppose $B$ is equal to the specimen radius and then assume a value for $R$ according to the configuration shown in Figure 6.1.

2. Then obtain the value of radial effective stress $\sigma'_r$ at the elastic-plastic boundary and consequently $q_i$ and $p'_i$ using either Equation (6.14) or Equation (6.15), depending on the boundary condition being considered, by insisting that the stress components satisfy the yield function expressed by Equation (5.16). Obtain values of specific volume $v_i$, elastic bulk modulus $K_i$ and elastic shear modulus $G_i$ on the elastic-plastic boundary from incremental elastic stress-strain relations (Equation (6.9) and Equation (6.10)). Find $\bar{w}_i$ by differentiating displacement $u$ with respect to time and substituting $r = R$ to give $\bar{w}_i = \dot{u}/\dot{R}$.

3. A differential equation solver is then used to evaluate $p', q, v, \bar{w}$ and $p'_c$ at any particular value of $\eta$ throughout the plastic region as an initial value problem. $p'_i, q_i, v_i, \bar{w}_i$ and $p'_c$ are the initial values at $\eta = 1$. 
4. Then evaluate the cavity radius $c$ using the value of $\eta$ ($= c/R$) at the cavity wall.

5. If the ratio $B/c$ is different to the desired ratio of chamber to cone radius, the initial starting $R$ value should be adjusted and steps 2 to 4 should be repeated. In general, if the ratio of $B/c$ is larger than desired value, the assumed $R$ value should be increased before repeating the steps, and vice versa.

Note that the results generated are relevant only at the instant when the cavity has expanded to a certain size corresponding to a certain ratio between $c$, $R$ and $B$. The results do not represent the stress history of the soil at the cavity wall during expansion, as they do when cavity expansion occurs in an infinite soil mass (as shown by Collins and Stimpson, 1994).

For the case of an infinite medium ($B \to \infty$) the differential equations become identical to those that would be derived using the similarity technique. Similarity exists when $B \to \infty$, meaning the expanding cavity from zero initial radius has no characteristic size and the deformation and stress configuration around the expanding cavity remain self-similar (Collins and Wang Yan, 1990; Collins et al., 1992; Collins and Stimpson, 1994). Also by applying $B \to \infty$ to either Equation (6.14) or (6.15) the solution for the cavity expansion problem in an infinite elastic medium is obtained. Note that $u \to 0$ as $r \to \infty$ and it can be shown that the infinite elastic region maintains zero volumetric strain, despite the material being compressible, therefore $p'$, $K$ and $G$ are constant in the infinite elastic region (Carter et al., 1986; Collins and Stimpson, 1994).

6.5 ERROR ANALYSIS

The error associated with neglecting a variables dependence on $\xi = B/R$ when expressing it in incremental form is investigated here. To do this it is supposed that drained cavity expansion occurs in a purely frictional Mohr-Coulomb soil, as closed form expressions are obtainable for when both $\xi$’s influence is neglected and correctly incorporated.
6.5.1 Exact solutions using the Mohr-Coulomb constitutive model

6.5.1.1 Stresses in the elastic region

The stresses in the elastic region obey Equations (6.14) and (6.15) for two different boundary conditions.

6.5.1.2 Stresses in the elastic-plastic region

The yield function takes the form:

\[ \sigma'_r = N\sigma'_o \]  \hspace{1cm} (6.31)

where

\[ N = \frac{1 + \sin \phi'}{1 - \sin \phi'} \]  \hspace{1cm} (6.32)

and \( \phi' \) is the internal angle of friction – a material constant. The soil is assumed to dilate at a constant rate. A single equation for the flow rule for both cylindrical and spherical cavities is:

\[ \frac{\dot{\varepsilon}_\theta}{\dot{\varepsilon}_r} = -\frac{k}{M} \]  \hspace{1cm} (6.33)

where

\[ M = \frac{1 + \sin \psi}{1 - \sin \psi} \]  \hspace{1cm} (6.34)

and \( \psi \) is the dilation angle – another material constant.

The radial stress \( \sigma'_r \) and the absolute value of tangential strain \( \varepsilon_T \) at the elastic-plastic boundary are then (Salgado, 1993):

\[ \sigma'_r = \frac{N(k + 1)}{N + k} p'_0 \frac{1}{1 + \gamma_s^{-(k+1)}} \]  \hspace{1cm} (6.35)
\[
\varepsilon_T = \frac{N-1}{N + k} \frac{p'_0}{2G} \frac{1 + \delta \xi^{-(k+1)}}{1 + \gamma \xi^{-(k+1)}}
\]

(6.36)

in which \(\gamma\) and \(\delta\) are constant and depend on the boundary conditions and material parameters. For the constant radial stress boundary condition:

\[
\gamma = \frac{k(N-1)}{N + k}
\]

(6.37)

and for the zero displacement boundary condition:

\[
\gamma = -\frac{N-1}{N + k} \frac{1 + (k-1)\mu}{1 - 2\mu}
\]

(6.39)

\[
\delta = -1
\]

(6.40)

Everywhere throughout the elastic-plastic region the equilibrium condition (Equation (6.12)) and the Mohr-Coulomb yield criterion (Equation (6.31)) must be satisfied:

\[
\frac{d\sigma_r'}{d\eta} + k \frac{N-1}{N} \frac{\sigma_r'}{\eta} = 0
\]

(6.41)

The distribution of radial stress throughout the elastic-plastic region is then:

\[
\sigma_r' = \sigma'_r \eta^{\beta-1}
\]

(6.42)

where:

\[
\beta = 1 - k \frac{N-1}{N}
\]

(6.43)
6.5.1.3 Displacement analysis

The constitutive relation in the elastic-plastic region then reduces to the following differential equation (Carter et al., 1986 and Salgado, 1993):

$$\dot{\varepsilon} + \alpha \frac{w}{\eta} = -\chi \frac{R \dot{\sigma}_r}{2G}$$

(6.44)

where

$$\chi = \frac{k(1-\mu)-k\mu(M+N)+[(k-2)\mu+1]MN}{[(k-1)\mu+1]MN}$$

(6.45)

in which $\alpha = \frac{k}{M}$. By substituting Equation (6.35) into Equation (6.42) it can be shown that, when $B$ is constant, the rate form of the radial stress becomes (Salgado, 1993):

$$\dot{\sigma}_r = \frac{W}{R} (1-\beta) \sigma_r \left(1 - \gamma \frac{\beta + k}{1+\gamma} \xi^{-(k+1)}\right) \eta^{\beta-1}$$

(6.46)

Substitution of Equation (6.46) into Equation (6.44) gives:

$$\dot{\varepsilon} + \alpha \frac{\tilde{w}}{\eta} = -\chi \eta^{\beta+1} k(1+1) \varepsilon_T \left(1 - \gamma \frac{\beta + k}{1+\beta} \xi^{-(k+1)}\right) \left(1 + \delta \xi^{-(k+1)}\right)$$

(6.47)

To integrate Equation (6.47) and find the complete expression for $\tilde{w}$ in the elastic-plastic region a boundary condition is needed. The velocity of the material at the elastic-plastic boundary will be used as this condition, and to obtain it, the displacement in the elastic region is firstly defined (Salgado, 1993):
The expression for $\tilde{w}$ in the elastic region then becomes:

$$\tilde{w} = (k+1) \frac{N-1}{N+k} \frac{p_0' \eta^{-k} + \eta \delta \xi^{-(k+1)}}{2G \left[1 + \gamma \xi^{-(k+1)}\right]^2}$$  \hspace{1cm} (6.49)

and at the elastic-plastic boundary is found by substituting $\eta = 1$:

$$\tilde{w}_{(\eta=1)} = \frac{(k+1)\varepsilon_r}{1 + \gamma \xi^{-(k+1)}}$$  \hspace{1cm} (6.50)

The integration of Equation (6.47), found using Equation (6.50), leads to the expression for $\tilde{w}$ in the elastic-plastic region:

$$\tilde{w} = \varepsilon_r \left[\hat{T} \eta^{-\alpha} - \hat{Z} \eta^{\beta}\right]$$  \hspace{1cm} (6.51)

in which

$$\hat{T} = \frac{(k+1)}{1 + \gamma \xi^{-(k+1)}} \left[1 + \frac{k\chi}{\alpha + \beta} \frac{1 - \gamma (\beta + k) \xi^{-/(k+1)}}{1 - \beta \xi^{-(k+1)}}\right]$$  \hspace{1cm} (6.52)

$$\hat{Z} = \frac{(k+1)}{1 + \gamma \xi^{-(k+1)}} \frac{k\chi}{\alpha + \beta} \left[1 - \frac{\gamma (\beta + k) \xi^{-/(k+1)}}{1 - \beta \xi^{-(k+1)}}\right]$$  \hspace{1cm} (6.53)

6.5.1.4 Limiting conditions at the cavity wall

An expression for the radial pressure at the cavity wall $\sigma_r'$ can be obtained from Equation (6.42) by substituting $\eta = \eta_c$ (where subscript $c$ denotes the cavity radius):
\[ \sigma'_c = \sigma'_h \eta_c^{\beta^{-1}} \] (6.54)

A limiting condition is established when there is no change in \( \sigma'_c \) as the cavity expansion occurs. Noting that \( \sigma'_h \) changes as the cavity expands in a finite medium, the limiting condition requires:

\[ d\sigma'_c = W \frac{\partial \sigma'_c}{\partial R} + w_c \frac{\partial \sigma'_c}{\partial c} = 0 \] (6.55)

Hence:

\[ \tilde{w}_c = \eta_c \left[ 1 - \frac{\gamma(k+1) \xi^{-(k+1)}}{1 - \beta} \right] \] (6.56)

Combining Equation (6.56) and Equation (6.51) gives:

\[ \frac{\varepsilon_T [\hat{T} \eta_c^{-(1+\alpha)} - \hat{Z} \eta_c^{\beta^{-1}}]}{\gamma(k+1) \xi^{-(k+1)}} \frac{1 - \beta}{1 + \gamma \xi^{-(k+1)}} = 1 \] (6.57)

For a cavity expanding in a soil of finite radial extent the limiting condition is attained when Equation (6.57) is satisfied, enabling \( \sigma'_c \) and other variables at the cavity wall to be evaluated.

The radial stress at the cavity wall is then found from Equation (6.54) using a value of \( \eta_c \) which satisfies Equation (6.57).
6.5.2 Approximate solutions using the Mohr-Coulomb constitutive model

Equations (6.31) to (6.45) also apply in this approximate analysis, but differences begin to emerge once \( \sigma'_r \) is presented in rate form. The approximate expression for the rate form of \( \sigma'_r \) is found by neglecting the influence of the ratio \( \zeta = B/R \) on \( \sigma'_r \). In other words, \( \sigma'_r \) is assumed constant during the cavity expansion in this approximate analysis, and is equal to the value at the final instant of cavity expansion once the limiting conditions have been reached. It follows that:

\[
\dot{\sigma}'_r = \frac{W}{R} (1 - \beta) \sigma'_r \eta^{\beta - 1} \tag{6.58}
\]

Substituting Equation (6.35) into Equation (6.58) gives:

\[
\dot{\sigma}'_r = \frac{W}{2G} \frac{k(k + 1) \varepsilon_x}{1 + \delta \zeta^{-(k+1)}} \eta^{\beta - 1} \tag{6.59}
\]

Substituting Equation (6.59) into Equation (6.44) gives:

\[
\frac{\partial \tilde{w}}{\partial \eta} + \alpha \frac{\tilde{w}}{\eta} = - \chi k(k + 1) \varepsilon_x \frac{\eta^{\beta - 1}}{1 + \delta \zeta^{-(k+1)}} \tag{6.60}
\]

Imposing the boundary condition given in Equation (6.50), the following expression for \( \tilde{w} \) in the elastic-plastic region becomes:

\[
\tilde{w} = \varepsilon_x \left[ T \eta^{-\alpha} - Z \eta^{\beta} \right] \tag{6.61}
\]

where

\[
T = \frac{(k + 1)}{1 + \gamma \zeta^{-(k+1)}} \left[ \frac{1 + \frac{k \chi}{\alpha + \beta} \varepsilon_x}{1 + \delta \zeta^{-(k+1)}} \right] \tag{6.62}
\]

\[
Z = \frac{(k + 1)}{1 + \delta \zeta^{-(k+1)}} \frac{k \chi}{\alpha + \beta} \tag{6.63}
\]
If a limiting condition is reached there is no further change in value of $\sigma'_c$ and from Equation. (6.42) it follows that:

$$d(\eta_c) = 0 \quad (6.64)$$

Hence:

$$\tilde{\omega}_c = \eta_c \quad (6.65)$$

Combining Equation (6.65) and Equation (6.61) for the limiting condition leads to:

$$\varepsilon_T \left[ T \eta_c \eta_c^{-(1+\alpha)} - Z \eta_c^{\beta-1} \right] = 1 \quad (6.66)$$

The radial stress at the cavity wall is then found from Equation (6.54) using a value of $\eta_c$ which satisfies Equation (6.66).

### 6.5.3 Analysed error

Radial stresses $\sigma'_c$ at the walls of expanding spherical and cylindrical cavities for two boundary conditions were calculated using the approximate analysis and compared with values obtained using the exact analysis. The error was calculated using $1 + \text{Error} = \frac{\sigma'_{c,\text{Approx}}}{\sigma'_{c,\text{Exact}}}$.

A range of material parameters were considered along with a range of $\zeta = B/R$ ratios. A Poisson’s ratio of 0.3 was assumed to apply in all cases.

Tables 6.1 to 6.4 present the errors for the range of input parameters. Because the Mohr-Coulomb model is perfectly plastic the error calculated depends on the value of $\zeta$ only. (At any given point, the error is independent of $p'_0$, $G$, $\phi'$ and $\psi$). In general, it is found that the error increases as the ratio $1/\zeta$ increases. The corresponding values of $R/c$ ($= 1/\eta_c$) were also calculated and tabulated for completeness, assuming $\phi' = 40^\circ$ and $\psi = 20^\circ$. 

6-20
Table 6.1 Resulting error for the spherical cavity expanding in a finite medium subjected to constant stress boundary condition.

<table>
<thead>
<tr>
<th>R/B (1/ξ)</th>
<th>Error (%)</th>
<th>θ'</th>
<th>ψ</th>
<th>R/c (1/η_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>+ 0.5</td>
<td>40°</td>
<td>20°</td>
<td>8.16</td>
</tr>
<tr>
<td>0.3</td>
<td>+ 1.5</td>
<td>40°</td>
<td>20°</td>
<td>8.26</td>
</tr>
<tr>
<td>0.4</td>
<td>+ 3.5</td>
<td>40°</td>
<td>20°</td>
<td>8.43</td>
</tr>
<tr>
<td>0.5</td>
<td>+ 6.4</td>
<td>40°</td>
<td>20°</td>
<td>8.70</td>
</tr>
<tr>
<td>0.6</td>
<td>+ 10.0</td>
<td>40°</td>
<td>20°</td>
<td>9.10</td>
</tr>
</tbody>
</table>

Table 6.2 Resulting error for a cylindrical cavity expanding in a finite medium subjected to constant stress boundary condition.

<table>
<thead>
<tr>
<th>R/B (1/ξ)</th>
<th>Error (%)</th>
<th>θ'</th>
<th>ψ</th>
<th>R/c (1/η_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.1</td>
<td>+ 0.5</td>
<td>40°</td>
<td>20°</td>
<td>22.81</td>
</tr>
<tr>
<td>0.2</td>
<td>+ 1.8</td>
<td>40°</td>
<td>20°</td>
<td>23.16</td>
</tr>
<tr>
<td>0.3</td>
<td>+ 3.9</td>
<td>40°</td>
<td>20°</td>
<td>23.73</td>
</tr>
<tr>
<td>0.4</td>
<td>+ 6.6</td>
<td>40°</td>
<td>20°</td>
<td>24.60</td>
</tr>
<tr>
<td>0.5</td>
<td>+ 8.7</td>
<td>40°</td>
<td>20°</td>
<td>25.53</td>
</tr>
<tr>
<td>0.6</td>
<td>+ 10.7</td>
<td>40°</td>
<td>20°</td>
<td>26.74</td>
</tr>
</tbody>
</table>

Table 6.3 Resulting error for a spherical cavity expanding in a finite medium subjected to rigid boundary condition.

<table>
<thead>
<tr>
<th>R/B (1/ξ)</th>
<th>Error (%)</th>
<th>θ'</th>
<th>ψ</th>
<th>R/c (1/η_c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2</td>
<td>- 0.8</td>
<td>40°</td>
<td>20°</td>
<td>8.06</td>
</tr>
<tr>
<td>0.3</td>
<td>- 2.7</td>
<td>40°</td>
<td>20°</td>
<td>7.91</td>
</tr>
<tr>
<td>0.4</td>
<td>- 6.6</td>
<td>40°</td>
<td>20°</td>
<td>7.61</td>
</tr>
</tbody>
</table>
Table 6.4 Resulting error for a cylindrical cavity expanding in a finite medium subjected to rigid boundary condition.

<table>
<thead>
<tr>
<th>$R/B$</th>
<th>Error (%)</th>
<th>$\phi$</th>
<th>$\psi$</th>
<th>$R/c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$(1/\xi)$</td>
<td></td>
<td></td>
<td></td>
<td>$(1/\eta_c)$</td>
</tr>
<tr>
<td>0.1</td>
<td>- 1.3</td>
<td>$40^\circ$</td>
<td>$20^\circ$</td>
<td>22.39</td>
</tr>
<tr>
<td>0.15</td>
<td>- 3.0</td>
<td>$40^\circ$</td>
<td>$20^\circ$</td>
<td>22.04</td>
</tr>
<tr>
<td>0.2</td>
<td>- 5.5</td>
<td>$40^\circ$</td>
<td>$20^\circ$</td>
<td>21.51</td>
</tr>
<tr>
<td>0.25</td>
<td>- 9.0</td>
<td>$40^\circ$</td>
<td>$20^\circ$</td>
<td>20.83</td>
</tr>
</tbody>
</table>

The results also show that the approximate $\sigma'_c$ value for a constant stress boundary condition is always larger than the exact value. Conversely, the approximate $\sigma'_c$ value for the zero displacement boundary condition is always smaller than the exact value. For spherical cavities and for both boundary conditions considered the maximum error is less than 7% when $R/B < 0.4$, and reduces significantly as $R/B$ becomes smaller. For cylindrical cavities and the constant stress boundary condition the maximum error is less than 7% when $R/B < 0.4$. It is only for cylindrical cavities and the rigid boundary condition that the error is larger than 7% when $R/B < 0.4$.

6.6 RESULTS FOR DRAINED CAVITY EXPANSIONS IN SYDNEY SAND OF FINITE RADIAL EXTENT

Now results will be presented for a range of initial values, specifically, for initial mean effective stresses $p'_0 = 25\text{kPa}, 50\text{kPa}, 100\text{kPa}, 200\text{kPa}, 300\text{kPa}, 400\text{kPa}, 500\text{kPa}$; and initial specific volumes $v_0 = 1.629, 1.726, 1.823$ (corresponding to relative densities $D_r = 90\%, 60\%, 30\%$ respectively).

Figures 6.2 and 6.3 present results of drained cavity expansions in infinite sand media for created spherical and cylindrical cavities. The $p'_0 \sim \sigma'_c$ plane is used in the presentations, where $\sigma'_c$ represents the radial stress at the cavity wall (a limiting value). This representation is similar to that commonly used to present CPT results where
cone penetration resistance $q_c$ (which is analogous to $\sigma'_c$) appears on the horizontal axis and overburden stress (analogous to $p'_0$) appears on the vertical axis. The radial stress at the cavity wall increases as the mean effective stress increases and this is more significant for higher values of relative density. Also for a given initial mean effective stress the radial stress at the cavity wall increases as the relative density of the soil increases. The radial stress at the spherical cavity wall is generally larger than that at a cylindrical cavity wall.

Figures 6.4-6.9 present results of spherical and cylindrical cavity expansions in finite and infinite sand media for the constant stress boundary condition. Various $R/B$ ratios were considered. The results were presented in term of $R/B$ to show the effects of change in elastic-plastic radius on cavity pressure. For the CPTs conducted in calibration chambers $B$ is related to the soil specimen radius and $c$ is related to the cone radius and $R$ is the radius of the elastic-plastic region (Figure 6.1). It is observed that for a certain value of $B$, $\sigma'_c$ decreases as $R$ increases. The reduction in the $\sigma'_c$ value is more pronounced for higher values of relative density. Also, as expected, reducing the $R/B$ ratio causes the results to approach those for expansions in infinite media. The values of $\sigma'_c$ for the spherical cavity and cylindrical cavity expanded in finite media are very close to those of infinite media when the $R/B$ ratio is less than 0.2 and 0.1, respectively. The observed reduction in $\sigma'_c$ by increasing the $R/B$ ratio is generally more significant for cylindrical cavities.

In Figures 6.10-6.15, for a given $R/B$ ratio, $\sigma'_c$ is normalized against its value for expansions in infinite media (having the same values of $p'_0$ and $v_0$ prior to expansion). The normalized quantity, denoted by $\sigma'_c/\sigma'_{c,\infty}$, is plotted against the size ratio $B/c$. Contours of equal $R/B$ ratios are indicated. For the constant stress boundary condition, and for a given value of $p'_0$, the $\sigma'_c/\sigma'_{c,\infty}$ value increases and approaches unity as $B/c$ increases. For the zero displacement boundary condition, and for a given value of $p'_0$, the $\sigma'_c/\sigma'_{c,\infty}$ value reduces and approaches unity as $B/c$ increases. For both boundary conditions, the size effects are more pronounced for decreasing values of $p'_0$ and increasing values of $D_r$. 
For spherical cavity expansions and both boundary conditions, the boundary size effects are insignificant for values of $B/c$ larger than about 35. For the example of the constant stress boundary condition, given the initial relative density $D_r = 60\%$ and $p'_0 = 25kPa$, the values of $\sigma'_c/\sigma'_{c,\infty}$ are 0.980, 0.997, 0.999 for $B/c$ values of 32, 64 and 160 respectively.

For cylindrical cavity expansions and the constant stress boundary condition, the boundary size effects become insignificant for values of $B/c$ larger than about 140, and for the zero displacement boundary condition, even larger values of $B/c$ are required before the boundary size effects become insignificant.

Comparing the two sets of results highlights that the influences of a finite boundary are more significant for cylindrical than spherical cavity expansions. This is expected as cylindrical expansions require the surrounding soil to undergo a much greater volumetric reduction (for a given cavity radius) than for spherical expansions, and hence the proximity of the boundary $B$ has a greater influence on the results. For the example of the constant stress boundary condition and a cylindrical cavity, given the initial $D_r = 60\%$ and $p'_0 = 25kPa$, the value of $\sigma'_c/\sigma'_{c,\infty}$ is 0.76 for $B/c = 34$ compared to a value of about 0.98 for a spherical cavity.

Supposing that $q_c$ measured from a CPT is closely related to $\sigma'_c$ for a spherical cavity expansion, these results imply that a CPT performed within a calibration chamber and a sand specimen subjected to the constant stress boundary condition will yield $q_c$ values that will be influenced less by the chamber size than when a zero displacement boundary condition is used.

Also, for the constant stress boundary condition, as long as the ratio between chamber and cone radii is larger than about 30, and for relative densities larger than about 30% and confining stresses larger than about 50kPa, the finite size of the chamber will reduce $q_c$ by no more than 3% of the value that would be measured in an infinitely large chamber. In short, for constant stress boundary conditions, the influence of the finite size of the chamber can be assumed negligible in most cases, with errors having
the same order of magnitude as the scatter that is normally associated with $q_c$ readings, as long as the chamber radius is more than 30 times larger than the cone radius.

It is well known that cavity expansion results, especially correlations between $\sigma'_c$, $D_r$ and $p'_0$, are sensitive to the constitutive model and input parameters used (e.g. Yu and Houlsby, 1991; Russell and Khalili, 2002; Russell, 2004; Jiang and Sun, 2011). Consequently, the observations made here should be taken as true only for the sand and model adopted. It is expected that results for other sands, especially those with significantly different compressibility (captured through the elastic stiffness as well as through the slope of a critical state line in the $v \sim \ln p'$ plane), will result in similar trends to those observed here but with a slight horizontal shift in the $B/l_c \sim \sigma'_c/\sigma'_{c,\infty}$ plane.

6.7 SIMILARITIES BETWEEN DRAINED AND UNDRAINED EXPANSIONS

Russell (2004) and Russell and Khalili (2006b) conducted an extensive investigation of cavity expansion in unsaturated soils of infinite radial extent considering both constant suction (drained) and constant moisture content (undrained) conditions. They showed that for constant moisture content expansions in unsaturated soils it is possible for $S_r$ to increase and for the pressure required to expand a cavity to tend towards that of a saturated undrained soil, although in general there was a far greater likelihood that this would occur in clays than sands.

Furthermore, they showed that for Sydney sand (referred to as Kurnell sand in their work) the soil responses around expanding cavities under constant suction (drained) and constant moisture content (undrained) conditions were virtually indistinguishable for a given set of initial conditions except when: i) the initial suction value, $s_0$, was no more than 0.1kPa larger than air entry/explusion value, $s_e$; and ii) volumetric contraction occurred in the soil during cavity expansion. The indistinguishable responses were due to the interaction of Equations (5.32) and (5.36) and the SWCC being very flat for suction values most likely to be encountered in practice.
The close similarity between constant suction and constant moisture content responses for Sydney sand is very convenient and will be exploited in chapter 8. It permits interpretation of cavity expansion results assuming constant suction (drained) conditions prevail. It also permits interpretation of measured cone penetration resistances, obtained for fast penetration rates (in the order of 2cm/s) where locally at the cone tip moisture content will be constant, assuming constant suction equal to the initial or far field value.

6.8 CONCLUDING REMARKS
The governing equations and procedure for solving the cavity expansion problem in finite soil media have been presented, applicable to when the soil stress-strain behaviour is described using a conventional elastic-plastic constitutive model formulated in a critical state framework. A simplifying assumption was introduced when converting a variable from rate form to differential form permitting an analytically derived solution to be obtained. The errors associated with the simplification have been investigated and shown to be less than 10% for conditions of relevance.

Using the new procedure cavity expansion results were generated for a quartz sand of infinite and finite radial extent and when subjected to both constant stress and zero displacement boundary conditions. The results were presented in a number of planes to explore and highlight the influences of the boundary condition type and boundary extent.

The pressure at a spherical cavity wall is influenced less by the size of the finite soil than is the pressure at a cylindrical cavity wall. Also, the pressure at the cavity wall, for both cylindrical and spherical cavities, is influenced less by the finite size of the soil for the constant stress boundary condition than for the zero displacement boundary condition.
The solution procedure is relevant to both saturated and unsaturated sands when drained conditions prevail. Also, as undrained and drained cavity expansions in unsaturated sands cause virtually indistinguishable soil responses, and assuming cone penetration resistance is directly related to the pressure required to expand a cavity, the procedure may be applied directly to interpretation of boundary influences on cone penetration resistances measured in calibration chambers as will be discussed in chapter 7.
Figure 6.1 Cavity expansion in elastic-plastic finite medium.
Figure 6.2 Spherical cavity expansion results for Sydney sand in infinite medium.
Figure 6.3 Cylindrical cavity expansion results for Sydney sand in infinite medium.
Figure 6.4 Spherical cavity expansion results for Sydney sand in finite medium subjected to the constant stress boundary condition, $D_r = 30\%$. 
Figure 6.5 Spherical cavity expansion results for Sydney sand in finite medium subjected to the constant stress boundary condition, $D_r = 60\%$. 
Figure 6.6 Spherical cavity expansion results for Sydney sand in finite medium subjected to the constant stress boundary condition, $D_r = 90\%$. 

\[ \sigma'_c \text{ (kPa)} \]

\[ p'_0 \text{ (kPa)} \]
Figure 6.7 Cylindrical cavity expansion results for Sydney sand in finite medium subjected to the constant stress boundary condition, $D_c = 30\%$. 

- Infinite Medium
  ($B \rightarrow \infty$)
- $R/B = 0.2$
- $R/B = 0.4$
- $R/B = 0.6$
Figure 6.8 Cylindrical cavity expansion results for Sydney sand in finite medium subjected to the constant stress boundary condition, $D_r = 60\%$. 

\[ \sigma'_c \text{ (kPa)} \]

\[ P'_0 \text{ (kPa)} \]

Infinite Medium

\( B \rightarrow \infty \)

R/B = 0.2

R/B = 0.4

R/B = 0.6
Figure 6.9 Cylindrical cavity expansion results for Sydney sand in finite medium subjected to the constant stress boundary condition, $D_r = 90\%$. 
Figure 6.10 Constant stress and zero displacement boundary effects on spherical cavity expansion results for various initial mean effective stresses, $D_r = 30\%$.
Figure 6.11 Constant stress boundary effects on spherical cavity expansion results for various initial mean effective stresses, $D_r =60\%$.
Figure 6.12 Constant stress and zero displacement boundary effects on spherical cavity expansion results for various initial mean effective stresses, \( D_r = 90\% \).
Figure 6.13 Constant stress and zero displacement boundary effects on cylindrical cavity expansion results for various initial mean effective stresses, $D_r = 30\%$.
Figure 6.14 Constant stress boundary effects on cylindrical cavity expansion results for various initial mean effective stresses, $D_r = 60\%$. 

\[ \frac{\sigma'_{c}}{\sigma'_{c,0}} \]

\[ R/B = 0.5 \]

\[ R/B = 0.4 \]

\[ R/B = 0.3 \]

\[ R/B = 0.2 \]

\[ R/B = 0.1 \]

\[ R/B = 0.0 \]
Figure 6.15 Constant stress and zero displacement boundary effects on cylindrical cavity expansion results for various initial mean effective stresses, $D_r = 90\%$.
CHAPTER 7

CALIBRATION CHAMBER BOUNDARY EFFECTS AND CORRECTION OF EXPERIMENTAL RESULTS

7.1 INTRODUCTION

An important issue concerning calibration chamber testing is that the size of the specimen is limited and that the penetration resistance may be influenced by the boundary conditions imposed by the chamber. The specimen whilst being tested may not represent soils in a free field condition even when initial properties and stress states are identical to that of the free field.

Significant contributions to this issue are the experimental studies of Parkin and Lunne (1982) and Bellotti (1984). Further considerations through experimental observation have been given to the calibration chamber size effects by Parkin (1988), Been et al. (1988) and Ghionna and Jamiolkowski (1991). The general conclusion drawn from these studies is that for the boundary effects to become negligible, the ratio of specimen diameter to cone diameter ($R_D$) needs to be in excess of about 35 for loose sands and in
excess of 60 for dense sands. Lesser values of $R_D$ may result in significant reductions in the measured values of the cone resistance.

Explanations of the problem have focused mainly on the constraining effect of the chamber boundaries as the cavity is created in the soil by the penetrating cone. Like in chapter 6, Schnaid and Houlsby (1991) and Salgado et al. (1998) used cylindrical expansion analyses to explain the size effects in calibration chambers, although their analyses were limited to cylindrical cavity expansions.

An alternate explanation was put forward by Wesley (2002), who argued that the change in the vertical stresses arising from the downward force of the cone penetrometer during penetration may cause a reduction of cone resistance with decreasing chamber size.

Despite these efforts, there are no reliable methods for correction of the CPT data from a calibration chamber so it becomes equivalent to what would be observed in free field conditions or an infinitely large chamber.

In this chapter a more complete understanding of the problem is put forward. Both the constraining effect of specimen boundaries (derived from cavity expansion theory outlined in chapter 6) and the vertical stress state change within a calibration chamber are considered at the same time. It is shown that both the constraining effects of the boundaries and changes in vertical stress state noticeably influence the penetration resistance. An analytical procedure is presented that predicts size effects in good agreement with the experimental data reported in the literature. The analytical procedure is then used for correction of the CPT results (conducted in the calibration chamber) presented in chapter 4 so they resemble those which would be obtained in free field conditions.
7.2 AN ANALYTICAL PROCEDURE TO ACCOUNT FOR CALIBRATION CHAMBER SIZE EFFECTS

The calibration chambers which have been used in past experimental studies of the CPT are different in their dimensions, mechanisms used to control lateral, top and bottom boundary conditions, and their stress application systems (Ghionna and Jamiolkowski, 1991). The experimental CPT results considered here to demonstrate the suitability of the new analytical tool, which were conducted to explore the effects of cone to chamber diameter ratio, were obtained using a double wall type chamber with a flexible membrane enclosing the specimens (Chapman, 1974 & Bellotti et al., 1982). In this type of chamber CPTs are performed in cylindrical specimens of finite size when subjected to one of four boundary conditions as listed in Table 7.1.

Vertical stress is applied to the bottom of the specimen via the chamber piston raised by pressurized water and the lateral stress is applied by a water filled annular space surrounding the specimen. After deposition, the specimen is consolidated to the desired stress state. The $K_0$ consolidation is achieved by increasing the vertical stress in small steps and allowing no lateral strain up to the final consolidation stress.

Table 7.1 Boundary conditions in calibration chamber tests (Ghionna and Jamiolkowski, 1991).

<table>
<thead>
<tr>
<th>Type of Boundary Conditions</th>
<th>Lateral Boundary Condition</th>
<th>Top and Bottom Boundary Conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>BC1</td>
<td>Constant Stress</td>
<td>Constant Stress</td>
</tr>
<tr>
<td>BC2</td>
<td>No displacement</td>
<td>No displacement</td>
</tr>
<tr>
<td>BC3</td>
<td>No displacement</td>
<td>Constant Stress</td>
</tr>
<tr>
<td>BC4</td>
<td>Constant Stress</td>
<td>No displacement</td>
</tr>
</tbody>
</table>

To impose BC1 on a specimen (constant stress at top, bottom and lateral boundaries), which is of particular interest in this study as it was used to generate the results in
Chapter 7

Boundary effects

7.2.1 Differing stress states in the field and a calibration chamber

It is important to note that the stress state in a calibration chamber ahead (below) and behind (above) the tip of a cone penetrometer moving downward is fundamentally different from that in free field conditions. This has been discussed by Been et al. (1988) and Wesley (2002), and is illustrated in Figure 7.1. For field conditions the vertical stress behind the cone remains constant (equal to the overburden pressure), while ahead of the cone it increases by an amount governed by the magnitude of the penetration resistance. In a calibration chamber the vertical stress ahead of the cone is controlled by the constant stress applied at the base of the specimen, whereas behind the cone, according to force equilibrium, the soil is unloaded by an amount dependant on the magnitude of the penetration resistance. The average effective vertical stress behind the tip of the penetrating cone (\( \sigma'_{v} \)) is expressed as (Wesley, 2002):

\[
\sigma'_{v} = \sigma'_{b} - \frac{q_{c}}{(R_{D})^{2}}
\]  

(7.1)

where \( q_{c} \) is cone resistance, \( \sigma'_{b} \) is the applied effective vertical stress at the base of the specimen and \( R_{D} \) is the specimen to cone diameter ratio. It can be observed that by decreasing diameter ratio, the reduction in vertical stress becomes more significant. When interpreting calibration chamber test results, to maintain a relationship to field conditions, it is more appropriate to use \( \sigma'_{v} \) in any correlation with \( q_{c} \), as \( \sigma'_{v} \) is analogous to overburden pressure in the field.
7.2.2 The analytical procedure

An analytical procedure is now presented, based on cavity expansion is soils of finite radial extent (Pournaghiazar et al., 2011b) and accounting for differing stress states, that can explain the size effects observed in calibration chamber studies of the cone penetration test. It can also be used to convert cone resistances measured in chambers to values that would be observed in the free field.

The procedure links cavity expansion pressure to \( R_D \), and accounts for the vertical stress reduction in a simple iterative way. Central to the procedure is the assumption (Bishop et al., 1945):

\[
\frac{q_c}{q_{c,\infty}} = \frac{\sigma'_c}{\sigma'_{c,\infty}}
\]  

(7.2)

in which the limiting radial pressures at the walls of expanded cavities in finite and infinite soil masses are denoted using \( \sigma'_c \) and \( \sigma'_{c,\infty} \), respectively, and cone penetration resistances measured in finite sized chambers and the free field are denoted using \( q_c \) and \( q_{c,\infty} \), respectively.

The steps to be followed to predict the cone resistance that would be measured in a calibration chamber for a certain \( R_D \), housing a soil of known properties, provided that the cone resistance for the free field condition and a soil having the same properties is known, are:

1. A cavity expansion analysis is first conducted supposing the soil is of infinite extent having the same engineering properties of the test specimen. The mean effective stress used in the analysis is found using \( (\sigma'_v + 2\sigma'_h)/3 \), where \( \sigma'_v \) and \( \sigma'_h \) represent effective vertical and lateral stresses that would be applied in the chamber without making the reduction to \( \sigma'_v \) as described above. This enables determination of \( \sigma'_{c,\infty} \).
2. The cone resistance for the free field condition may be assumed equal to the cone resistance that would be measured in an infinitely large chamber \((q_{c,\infty})\). As indicated in Figures 7.2 and 7.3, the free field cone resistance is assumed to be the asymptotic (upper limit) value that would be observed as \(R_D\) increases to a very large value.

3. For a finite sized chamber, and a particular \(R_D\) value of interest (which is held constant throughout this procedure), it is assumed (temporarily) that the corresponding cone resistance measured in a calibration chamber is equal to the far field value (the errors associated with assuming \(q_c = q_{c,\infty}\) here will vanish after a number of iterations as explained below). A first estimate of the average vertical effective stress behind the cone in a calibration chamber may then be calculated (Equation 7.1) for the given \(R_D\) value.

4. The mean effective stress behind the cone is then recalculated using the reduced vertical stress (from step 3) and unaltered lateral stress.

5. The recalculated mean effective stress behind the cone from step 4 and the \(R_D\) value are then used in the analysis of an expanding cavity in a finite soil to determine a first estimate of \(\sigma_c'\). It is here that it can be seen how the cavity expansion analysis takes into account both vertical stress reduction and a lateral boundary of finite extent.

6. Now that values of \(\sigma_c'\), \(\sigma_{c,e}'\) and \(q_{c,e}\) have been established, a new value of \(q_c\) is calculated using Equation 7.2.

7. Iterations are performed by introducing this new \(q_c\) value at step 3 and reworking through to step 6.

8. After several iterations the \(q_c\) value determined at step 6 will approach a constant.
The steps to be followed to convert cone penetration resistance measured in a chamber for a certain $R_D$ to a free field value, when the soil in the chamber and the free field have the same properties and are known, are:

1. Solve the cavity expansion problem for the soil when of infinite extent having the same engineering properties of the test specimen to determine $\sigma'_c$. Note that the mean effective stress used in this step is found without making the reduction to $\sigma'_c$.

2. For the finite condition and relevant $R_D$ value, calculate the average vertical stress behind the cone from Equation (7.1) using the measured value of $q_c$ in the chamber.

3. Calculate the mean effective stress behind the cone using the reduced vertical stress (from step 2) and unaltered lateral stress.

4. The calculated mean effective stress behind the cone and the $R_D$ value are then used in the analysis of an expended cavity in finite media to determine $\sigma'_c$.

5. Now that values of $\sigma'_c$, $\sigma'_{c,\infty}$ and $q_c$ are available, the value of $q_{c,\infty}$ is calculated using Equation 7.2.

### 7.2.3 Demonstration and comparisons with data from the literature

The success of the procedure will be demonstrated by comparing normalized cavity expansion results to normalized cone penetration resistances obtained from calibration chamber tests with different chamber/cone diameter ratios. In particular, the results reported by Parkin and Lunne (1982) for Hokksund sand subjected to a vertical stress of 50 kPa (Figure 7.2) and by Bellotti (1984) for Ticino sand subjected to a vertical stress of 100 kPa (Figure 7.3) are considered. Furthermore, only the results from CPTs conducted in normally consolidated specimens subjected to boundary condition BC1 are considered as it was only for these tests that the horizontal stress states were known.
It is tempting to also use results in these studies from CPTs conducted in specimens subjected to the so called boundary condition BC2, although for these the lateral boundary movement was restrained in an average sense (the average lateral displacement was zero rather than the boundary being rigid) meaning localized lateral displacements were not prevented making the boundary condition quite different from a true BC2 rigid boundary condition (Salgado et al., 1998). For BC3 theory would predict calibration chamber qc values to increase with reducing RD values (as also observed in centrifuge tests reported by Bolton et al., 1999) whereas the results show the opposite to be the case.

The constitutive model presented in chapter 5 was assumed when undertaking the cavity expansion analyses. Based on the reported characteristics in the literature, Hokksund sand seems to be less compressible than Sydney sand. A value of $\kappa = 0.025$ (Equations 5.10 and 5.13) has therefore been assumed for Hokksund sand (compared to $\kappa = 0.006$ for Sydney sand) to account for the difference in compressibility.

Figure 7.4 presents a plot of diameter ratio $R_D$ against normalized cavity pressures ($\frac{\sigma'_v}{\sigma'_{c,v}}$) and normalized CPT results ($\frac{q_c}{q_{c,v}}$) of Parkin and Lunne (1982) for Hokksund sand with $D_r = 90\%$ and when the applied vertical stress at the base of the chamber was $\sigma'_v = 50$ kPa. More specifically, the solid symbols represent the experimental data. The dotted line represents results from spherical cavity expansion analysis ignoring the vertical stress reduction behind the cone and assuming $K_0 = \frac{\sigma'_h}{\sigma'_v} = 0.41$ for all diameter ratios. The solid line represents results from spherical cavity expansion analysis considering the vertical stress reduction behind the cone and again assuming $K_0 = 0.41$ for all diameter ratios. Free field cone resistance value was inferred from the tests reported with larger $R_D$ values. Since the tests with larger $R_D$ values were conducted under $K_0 = 0.41$, the same $K_0$ has been assumed for free field condition. However, as three of the experimental results had $K_0$ values that differed slightly from 0.41 (as indicated in Table 7.2), part of the observed size effects may be due to the difference in initial horizontal stress state. Therefore predictions for
$K_0 = 0.36$ and 0.47 are also shown (obtained using adjusted vertical stress). Note that the value of $\sigma'_{c,\infty}$ was derived assuming $K_0 = 0.41$ for all of the predictions presented. The results are also summarized in Table 7.2.

**Table 7.2** Adjusted values of vertical stress, normalized spherical cavity pressure and predicted cone resistance for tests conducted in Hokksund sand by Parkin and Lunne (1982) inferred from Figure 7.2 ($D_r = 90\%$, applied vertical stress, $\sigma'_v = 50$ kPa).

<table>
<thead>
<tr>
<th>$R_D$</th>
<th>Initial $K_0$ (kPa)</th>
<th>$\sigma'_h$ (kPa)</th>
<th>Corrected $\sigma'_v$ (kPa)</th>
<th>$\sigma'_m$ (kPa)</th>
<th>$\sigma'<em>c/\sigma'</em>{c,\infty}$</th>
<th>Predicted $q_c$ (MPa)</th>
<th>Measured $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field</td>
<td>0.41</td>
<td>20.5</td>
<td>50</td>
<td>30.33</td>
<td>1</td>
<td>17.5</td>
<td>17.5</td>
</tr>
<tr>
<td>70</td>
<td>0.41</td>
<td>20.5</td>
<td>46.56</td>
<td>29.19</td>
<td>0.96</td>
<td>16.8</td>
<td>—</td>
</tr>
<tr>
<td>60</td>
<td>0.41</td>
<td>20.5</td>
<td>45.38</td>
<td>28.79</td>
<td>0.95</td>
<td>16.63</td>
<td>—</td>
</tr>
<tr>
<td>48.3</td>
<td>0.41</td>
<td>20.5</td>
<td>43.1</td>
<td>28.03</td>
<td>0.92</td>
<td>16.1</td>
<td>16</td>
</tr>
<tr>
<td>34.2</td>
<td>0.47</td>
<td>23.5</td>
<td>36.83</td>
<td>27.94</td>
<td>0.88</td>
<td>15.4</td>
<td>15</td>
</tr>
<tr>
<td>30.2</td>
<td>0.36</td>
<td>18</td>
<td>35.42</td>
<td>23.81</td>
<td>0.76</td>
<td>13.3</td>
<td>11.5</td>
</tr>
<tr>
<td>21.4</td>
<td>0.36</td>
<td>18</td>
<td>27.07</td>
<td>21.02</td>
<td>0.6</td>
<td>10.5</td>
<td>7.5</td>
</tr>
</tbody>
</table>

Figure 7.5 presents a plot of diameter ratio $R_D$ against normalized cavity pressures ($\sigma'_c/\sigma'_{c,\infty}$) and normalized CPT results ($q_c/q_{c,\infty}$) of Parkin and Lunne (1982) for Hokksund sand with $D_r = 30\%$ when the applied vertical stress at the base of the chamber was $\sigma'_v = 50$ kPa. The dotted line represents results from spherical cavity expansion analysis ignoring the vertical stress reduction behind the cone and assuming $K_0 = 0.5$ for all diameter ratios. The solid line represents results from spherical cavity expansion analysis considering the vertical stress reduction behind the cone and again assuming $K_0 = 0.5$ for all diameter ratios. The results are also summarized in Table 7.3.
Table 7.3 Adjusted values of vertical stress, normalized spherical cavity pressure and predicted cone resistance for tests conducted in Hokksund sand by Parkin and Lunne (1982) inferred from Figure 7.2 ($D_r = 30\%$, applied vertical stress, $\sigma'_v = 50$ kPa).

<table>
<thead>
<tr>
<th>$R_D$</th>
<th>initial $K_0$</th>
<th>$\sigma'_h$ (kPa)</th>
<th>Corrected $\sigma'_v$ (kPa)</th>
<th>$\sigma'_m$ (kPa)</th>
<th>$\sigma'<em>v/\sigma'</em>{c,\infty}$</th>
<th>Predicted $q_c$ (MPa)</th>
<th>Measured $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field</td>
<td>0.5</td>
<td>25</td>
<td>50</td>
<td>33.33</td>
<td>1</td>
<td>3.8</td>
<td>3.8</td>
</tr>
<tr>
<td>70</td>
<td>0.5</td>
<td>25</td>
<td>49.23</td>
<td>33.08</td>
<td>0.99</td>
<td>3.8</td>
<td>—</td>
</tr>
<tr>
<td>50</td>
<td>0.5</td>
<td>25</td>
<td>48.51</td>
<td>32.84</td>
<td>0.98</td>
<td>3.7</td>
<td>—</td>
</tr>
<tr>
<td>40</td>
<td>0.5</td>
<td>25</td>
<td>47.70</td>
<td>32.57</td>
<td>0.97</td>
<td>3.7</td>
<td>—</td>
</tr>
<tr>
<td>30.2</td>
<td>0.5</td>
<td>25</td>
<td>46.08</td>
<td>32.03</td>
<td>0.94</td>
<td>3.6</td>
<td>3.4</td>
</tr>
<tr>
<td>21.4</td>
<td>0.5</td>
<td>25</td>
<td>43.11</td>
<td>31.04</td>
<td>0.83</td>
<td>3.2</td>
<td>3</td>
</tr>
</tbody>
</table>

Figure 7.6 presents a plot of diameter ratio $R_D$ against normalized cavity pressures ($\sigma'_v/\sigma'_{c,\infty}$) and normalized CPT results ($q_c/q_{c,\infty}$) of Bellotti (1984) for Ticino sand with $D_r = 90\%$ when the applied vertical stress at the base of the chamber was $\sigma'_v = 100$ kPa. The dotted line represents results from spherical cavity expansion analysis ignoring the vertical stress reduction behind the cone and assuming $K_0 = 0.4$ for all diameter ratios. The solid line represents results from spherical cavity expansion analysis considering the vertical stress reduction behind the cone and again assuming $K_0 = 0.4$ for all diameter ratios. The results are also summarized in Table 7.4.

Results derived using cylindrical cavity expansion analyses are also shown in Figures 7.4 to 7.6. It is observed that the cylindrical cavity expansion analysis gives poor predictions of boundary influences on cone resistance in a calibration chamber for loose specimens. The vertical stress reduction is not needed in the cylindrical cavity expansion analysis due to the two dimensional plane strain nature of the problem.
Figures 7.4 - 7.6 show that incorporating the vertical stress state change into the spherical cavity expansion analysis enables appropriate size effect predictions which are in good agreement with the experimental results for either loose or dense specimens.

**Table 7.4** Adjusted values of vertical stress, normalized spherical cavity pressure and predicted cone resistance for tests conducted in Ticino sand by Bellotti (1984) ($D_r = 90\%$, $\sigma'_v = 100$ kPa).

<table>
<thead>
<tr>
<th>$R_D$</th>
<th>initial $K_0$</th>
<th>$\sigma'_b$ (kPa)</th>
<th>Corrected $\sigma'_v$ (kPa)</th>
<th>$\sigma'_m$ (kPa)</th>
<th>$\sigma'<em>v/\sigma'</em>{c,\infty}$</th>
<th>Predicted $q_c$ (MPa)</th>
<th>Measured $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field</td>
<td>0.4</td>
<td>40</td>
<td>100</td>
<td>60.00</td>
<td>1</td>
<td>21</td>
<td>21</td>
</tr>
<tr>
<td>70</td>
<td>0.4</td>
<td>40</td>
<td>95.80</td>
<td>58.60</td>
<td>0.98</td>
<td>20.58</td>
<td>—</td>
</tr>
<tr>
<td>60</td>
<td>0.4</td>
<td>40</td>
<td>94.34</td>
<td>58.11</td>
<td>0.97</td>
<td>20.37</td>
<td>20.5</td>
</tr>
<tr>
<td>48</td>
<td>0.4</td>
<td>40</td>
<td>91.34</td>
<td>57.11</td>
<td>0.95</td>
<td>19.95</td>
<td>20</td>
</tr>
<tr>
<td>40</td>
<td>0.4</td>
<td>40</td>
<td>87.79</td>
<td>55.93</td>
<td>0.93</td>
<td>19.53</td>
<td>—</td>
</tr>
<tr>
<td>32</td>
<td>0.4</td>
<td>40</td>
<td>81.75</td>
<td>53.92</td>
<td>0.89</td>
<td>18.69</td>
<td>19</td>
</tr>
</tbody>
</table>

Using the procedure presented here it is possible to quantify how much the constraining effect of the lateral boundary and the change of stress state behind the cone contribute to the reduction in cone penetration resistance. The individual contributions are summarized in Tables 7.5 and 7.6, relevant to tests conducted by Parkin and Lunne (1982) in Hokksund sand with $D_r = 90\%$ and $D_r = 30\%$, respectively. It can be seen that the change of stress state contribution to the reduction of penetration resistance increases with specimen density. Also, contribution of the constraining effect is small for $R_D$ ratios larger than about 30, although increases drastically for $R_D$ values less than 30.

For the example of a specimen with $D_r = 90\%$ and $R_D = 30.2$ (Table 7.5), when the total reduction in penetration resistance was 21\%, the constraining effect contributed to a reduction of 6.2\% and the change of stress state contributed to a reduction of 14.8\%.
For the example of a specimen with $D_r = 90\%$ and $R_D = 21.4$, when the total reduction in cone penetration resistance was 37\%, the constraining effect contributed to a reduction of 17\% and the change of stress state contributed to a reduction of 20\%. For the example of a specimen with $D_r = 30\%$ and $R_D = 21.4$ (Table 7.6), when the total reduction in penetration resistance was 17\%, the constraining effect contributed to a reduction of 10.5\% and the change of stress state contributed to a reduction of 6.5\%.

**Table 7.5** Individual contributions of the constraining effect and the change of stress state behind the cone for calibration chamber tests conducted by Parkin and Lunne (1982), based on spherical cavity pressure (Hokksund sand $D_r = 90\%$, applied vertical stress of 50 kPa, $K_0 = 0.41$).

<table>
<thead>
<tr>
<th>$R_D$</th>
<th>$\sigma'_c/\bar{\sigma}_c$ (corrected $\sigma_c$)</th>
<th>Total reduction in cone resistance (%)</th>
<th>Contribution of finite boundary (%)</th>
<th>Contribution of change in stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>0.96</td>
<td>4.0</td>
<td>0.5</td>
<td>3.5</td>
</tr>
<tr>
<td>48.3</td>
<td>0.92</td>
<td>8.0</td>
<td>1.5</td>
<td>6.5</td>
</tr>
<tr>
<td>34.2</td>
<td>0.84</td>
<td>16.0</td>
<td>4.0</td>
<td>12.0</td>
</tr>
<tr>
<td>30.2</td>
<td>0.79</td>
<td>21.0</td>
<td>6.2</td>
<td>14.8</td>
</tr>
<tr>
<td>21.4</td>
<td>0.63</td>
<td>37.0</td>
<td>17.0</td>
<td>20.0</td>
</tr>
</tbody>
</table>

Tabulated in Table 7.7 are contributions by the constraining effect and change of stress state to the reduction of penetration resistance for tests conducted by Bellotti (1984) in Ticino sand with $D_r = 90\%$. It can be seen that the change of stress state most noticeably contributes to the reduction of cone resistance, while that of the constraining effect is small for the relevant $R_D$ ratios. For the example of a test with $R_D = 32$, when the total reduction in cone penetration resistance was 11\%, the constraining effect contributed to a reduction of 2\% and the change of stress state contributed to a reduction of 9\%.
Table 7.6  Individual contributions of the constraining effect and the change of stress state behind the cone to the reduction in cone resistance for calibration chamber tests conducted by Parkin and Lunne (1982), based on spherical cavity pressure (Hokksund sand $D_r = 30\%$, applied vertical stress of 50 kPa, $K_0 = 0.5$).

<table>
<thead>
<tr>
<th>$R_D$</th>
<th>$\sigma'<em>c/\sigma'</em>{c,\infty}$ (corrected $\sigma_c$)</th>
<th>Total reduction in cone resistance (%)</th>
<th>Contribution of finite boundary (%)</th>
<th>Contribution of change in stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>0.99</td>
<td>1.0</td>
<td>0.4</td>
<td>0.6</td>
</tr>
<tr>
<td>50</td>
<td>0.98</td>
<td>2.0</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>40</td>
<td>0.97</td>
<td>3.0</td>
<td>1.6</td>
<td>1.4</td>
</tr>
<tr>
<td>30.2</td>
<td>0.94</td>
<td>6.0</td>
<td>3.5</td>
<td>2.5</td>
</tr>
<tr>
<td>21.4</td>
<td>0.83</td>
<td>17.0</td>
<td>10.5</td>
<td>6.5</td>
</tr>
</tbody>
</table>

More generally, the influence of the constraining effect of a finite sized boundary is more pronounced for expansions in dense sands. Also its influence decreases as the confining stress increases. The influence due to the reduction in the vertical stress behind the cone is most significant for high values of cone resistance and therefore is most pronounced for dense specimens. These theoretical results confirm that size effects are more noticeable for cone penetration tests performed in dense sands. The initial values of horizontal and vertical stress, and their relative magnitude controlled through $K_0$, also influence the severity of the observed size effects. The size effects become less significant in loose sands. Specifically, it is observed that for a relative density of 30% and vertical stress of 50 kPa, size effects are negligible for diameter ratios of more than 35.
Table 7.7  Individual contributions of the constraining effect and the change of stress state behind the cone to the reduction in cone resistance for calibration chamber tests conducted by Bellotti (1984) based on spherical cavity pressure (Ticino sand, $D_r = 90\%$, applied vertical stress of 100 kPa, $K_0 = 0.4$).

<table>
<thead>
<tr>
<th>$R_D$</th>
<th>$\sigma_c'/\sigma_c'$ (corrected $\sigma_c$)</th>
<th>Total reduction in cone resistance (%)</th>
<th>Contribution of finite boundary (%)</th>
<th>Contribution of change in stress (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>0.98</td>
<td>2.0</td>
<td>0.2</td>
<td>1.8</td>
</tr>
<tr>
<td>60</td>
<td>0.97</td>
<td>3.0</td>
<td>0.3</td>
<td>2.7</td>
</tr>
<tr>
<td>48</td>
<td>0.95</td>
<td>5.0</td>
<td>0.6</td>
<td>4.4</td>
</tr>
<tr>
<td>40</td>
<td>0.93</td>
<td>7.0</td>
<td>1.0</td>
<td>6.0</td>
</tr>
<tr>
<td>32</td>
<td>0.89</td>
<td>11.0</td>
<td>2.0</td>
<td>9.0</td>
</tr>
</tbody>
</table>

7.3 CORRECTION OF EXPERIMENTAL RESULTS

The procedure presented in section 7.2 will now be used to infer the free field cone resistance corresponding to the CPTs conducted within the calibration chamber (presented in chapter 4). As the CPTs were conducted in sand specimens with diameter of 460 mm subjected to constant stress boundary condition, using a cone with diameter of 16 mm, a value of $R_D = 28.75$ applies to all of the tests performed in the calibration chamber.

Calibration chamber size effects for the CPTs conducted in saturated and unsaturated Sydney sand with initial relative densities of $D_r = 33\%$ and $D_r = 61\%$ are summarized in Tables 7.8 and 7.9, respectively. The corresponding estimated free field cone resistances are also tabulated. The confining stress values for unsaturated tests refer to the net stress instead of the effective stress used in chapter 6.

The maximum predicted size effect is less than 8% and 7% for the tests conducted in specimens with relative density of $D_r = 61\%$ and 33%, respectively.
As mentioned in section 7.2 the initial values of horizontal and vertical stresses, and their relative magnitude controlled through $K_0$, may influence the severity of the observed size effects. However, as the tests were conducted in sand specimens subjected to isotropic confining stress, the influence of the vertical stress change have been less prominent. Furthermore, the size effects are more pronounced for dense specimens and decrease as the confining stress increase. Also, for saturated and unsaturated specimens with identical initial relative densities and constant net confining stress, the constraining effect of the boundary is constant but the total size effects were slightly more pronounced for the unsaturated specimens due to the effect of stress state change. For the tests subjected to identical net confining stresses the effects of stress state change increased as measured values of cone resistance increased with suction.

Now the corrected saturated test results are compared with the established CPT correlations to assess the reliability of the apparatus developed, testing procedures and theoretical corrections. Using the correlations of Schmertmann (1976), a relative density of $D_r=31\%$ is obtained for loose specimens (prepared relative density of $D_r=33\%$) whereas the correlations of Baldi et al. (1986) obtain a relative density of $35\%$ for this set of specimens. For medium dense specimens (prepared relative density of $D_r=61\%$), Schmertmann (1976) and Baldi et al. (1986) correlations obtain a relative density of $57\%$ and $63\%$ respectively. Considering the sensitivity of CPT correlations to sand compressibility, it can be concluded that the testing procedures and theoretical corrections provide robust set of data. Using the correlations of Robertson and Campanella (1983), a peak friction angle of $34^\circ$ and $38^\circ$ is obtained for specimens with relative densities of $33\%$ and $61\%$ respectively.
Table 7.8 Calibration chamber size effects and corrected values of cone resistance for the CPTs conducted in saturated and unsaturated Sydney sand with an initial relative density of $D_r = 33\%$.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Isotropic net stress (kPa)</th>
<th>Measured $q_c$ (MPa)</th>
<th>Corrected $\sigma_v$ (kPa)</th>
<th>Corrected $\sigma_m$ (kPa)</th>
<th>$\sigma_c/\sigma_{c,\infty}$</th>
<th>Corrected $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL25</td>
<td>25.0</td>
<td>2.0</td>
<td>22.58</td>
<td>24.19</td>
<td>0.955</td>
<td>2.09</td>
</tr>
<tr>
<td>UL25-7-13</td>
<td>25.0</td>
<td>2.7</td>
<td>21.73</td>
<td>23.91</td>
<td>0.946</td>
<td>2.85</td>
</tr>
<tr>
<td>UL25-50</td>
<td>25.0</td>
<td>3.4</td>
<td>20.89</td>
<td>23.63</td>
<td>0.937</td>
<td>3.63</td>
</tr>
<tr>
<td>UL25-200</td>
<td>25.0</td>
<td>3.8</td>
<td>20.40</td>
<td>23.47</td>
<td>0.931</td>
<td>4.08</td>
</tr>
<tr>
<td>SL50</td>
<td>50.0</td>
<td>3.4</td>
<td>45.89</td>
<td>48.63</td>
<td>0.964</td>
<td>3.53</td>
</tr>
<tr>
<td>SL50R</td>
<td>50.0</td>
<td>3.3</td>
<td>46.01</td>
<td>48.67</td>
<td>0.964</td>
<td>3.42</td>
</tr>
<tr>
<td>UL50-25</td>
<td>50.0</td>
<td>4.2</td>
<td>44.92</td>
<td>48.31</td>
<td>0.959</td>
<td>4.38</td>
</tr>
<tr>
<td>UL50-200</td>
<td>50.0</td>
<td>5.1</td>
<td>43.83</td>
<td>47.94</td>
<td>0.953</td>
<td>5.35</td>
</tr>
<tr>
<td>UL50-200R</td>
<td>50.0</td>
<td>5.3</td>
<td>43.59</td>
<td>47.86</td>
<td>0.953</td>
<td>5.56</td>
</tr>
<tr>
<td>SL100</td>
<td>100.0</td>
<td>5.8</td>
<td>92.98</td>
<td>97.66</td>
<td>0.972</td>
<td>5.97</td>
</tr>
<tr>
<td>UL100-25</td>
<td>100.0</td>
<td>6.6</td>
<td>92.02</td>
<td>97.34</td>
<td>0.970</td>
<td>6.80</td>
</tr>
<tr>
<td>UL100-25R</td>
<td>100</td>
<td>6.5</td>
<td>92.14</td>
<td>97.38</td>
<td>0.970</td>
<td>6.70</td>
</tr>
<tr>
<td>UL100-200</td>
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<td>7.6</td>
<td>90.81</td>
<td>96.94</td>
<td>0.967</td>
<td>7.86</td>
</tr>
</tbody>
</table>
Table 7.9 Calibration chamber size effects and corrected values of cone resistance for the CPTs conducted in saturated and unsaturated Sydney sand with an initial relative density of $D_r = 61\%$.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Isotropic net stress (kPa)</th>
<th>Measured $q_c$ (MPa)</th>
<th>Corrected $\sigma_v$ (kPa)</th>
<th>Corrected $\sigma_m$ (kPa)</th>
<th>$\sigma_c/\sigma_{c,\infty}$</th>
<th>Corrected $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SM30</td>
<td>30</td>
<td>3.1</td>
<td>26.25</td>
<td>28.75</td>
<td>0.939</td>
<td>3.30</td>
</tr>
<tr>
<td>UM30-25</td>
<td>30</td>
<td>3.9</td>
<td>25.28</td>
<td>28.43</td>
<td>0.931</td>
<td>4.19</td>
</tr>
<tr>
<td>UM30-200</td>
<td>30</td>
<td>4.8</td>
<td>24.19</td>
<td>28.06</td>
<td>0.921</td>
<td>5.21</td>
</tr>
<tr>
<td>SM50</td>
<td>50</td>
<td>5</td>
<td>43.95</td>
<td>47.98</td>
<td>0.945</td>
<td>5.29</td>
</tr>
<tr>
<td>UM50-25</td>
<td>50</td>
<td>5.8</td>
<td>42.98</td>
<td>47.66</td>
<td>0.940</td>
<td>6.17</td>
</tr>
<tr>
<td>UM50-200</td>
<td>50</td>
<td>6.7</td>
<td>41.89</td>
<td>47.30</td>
<td>0.934</td>
<td>7.17</td>
</tr>
<tr>
<td>SM100</td>
<td>100</td>
<td>11</td>
<td>86.69</td>
<td>95.56</td>
<td>0.949</td>
<td>11.59</td>
</tr>
<tr>
<td>UM100-25</td>
<td>100</td>
<td>12.2</td>
<td>85.24</td>
<td>95.08</td>
<td>0.946</td>
<td>12.90</td>
</tr>
<tr>
<td>UM100-25R</td>
<td>100</td>
<td>12</td>
<td>85.48</td>
<td>95.16</td>
<td>0.947</td>
<td>12.67</td>
</tr>
<tr>
<td>UM100-200</td>
<td>100</td>
<td>13.7</td>
<td>83.43</td>
<td>94.48</td>
<td>0.94</td>
<td>14.57</td>
</tr>
<tr>
<td>SM150</td>
<td>150</td>
<td>18</td>
<td>128.22</td>
<td>142.74</td>
<td>0.951</td>
<td>18.93</td>
</tr>
<tr>
<td>UM150-200</td>
<td>150</td>
<td>20.2</td>
<td>125.56</td>
<td>141.85</td>
<td>0.947</td>
<td>21.33</td>
</tr>
</tbody>
</table>
7.4 CONCLUDING REMARKS

Using cavity expansion theory it is shown for CPTs conducted in specimens subjected to constant stress boundary conditions (BC1) within a calibration chamber that the size effects are related to both the reduction in vertical stress state behind the cone and the finite extent of the boundary. These two factors cause separate decreases in cone resistance as the diameter ratio decreases.

Predictions made by spherical cavity expansion analysis incorporating changes in vertical stress behind the penetrating cone are in good agreement with experimental results from the literature. Spherical cavity expansion analyses show that the finite extent of the specimen boundary causes influences that are more pronounced for expansions in dense sands. The reduction in the vertical stress behind the penetrating cone and consequently the reduction in mean stress behind the cone is significant for high values of cone resistance which are inherent of dense specimens. The initial values of horizontal and vertical stress, and their relative magnitude controlled through $K_0$, also influence the severity of the observed boundary effects. Repeating the analyses using cylindrical cavities provides a very poor fit to the data, indicating that spherical cavity expansion analysis is the more suitable tool for interpretation.

Size effects should be considered in the interpretation of the CPTs conducted in calibration chambers. A new technique has been presented making it possible to correct the results to a free field condition with good precision. The technique has been used to infer the free field values of cone resistance for the CPT results presented in chapter 4.
Chapter 7

Boundary effects

Figure 7.1 Stress state around a penetrating cone in (a) field and (b) calibration chamber (after Been et al., 1988).

Figure 7.2 Calibration chamber size effects on cone resistance for Hokksund sand, $\sigma'_v = 50$ kPa (after Parkin and Lunne, 1982).
Figure 7.3 Calibration chamber size effects on cone resistance for normally consolidated dense Ticino sand, $D_r = 90\%$, $\sigma' = 100$ kPa (after Bellotti, 1984).
Figure 7.4 Predicted boundary effects for calibration chamber tests conducted by Parkin and Lunne (1982) using cavity expansion theory, based on spherical cavity pressure, spherical cavity pressure taking account of changes in vertical stress & cylindrical cavity pressure (Hokksund sand, constant stress boundary, normally consolidated, $D_v = 90\%$, applied vertical stress of 50 kPa).
Figure 7.5 Predicted boundary effects for calibration chamber tests conducted by Parkin and Lunne (1982) using cavity expansion theory, based on spherical cavity pressure, spherical cavity pressure taking account of changes in vertical stress & cylindrical cavity pressure (Hokksund sand, constant stress boundary, normally consolidated, $D_r = 30\%$, applied vertical stress of 50 kPa).
Figure 7.6 Predicted boundary effects for calibration chamber tests conducted by Bellotti (1984) using cavity expansion theory, based on spherical cavity pressure, spherical cavity pressure taking account of changes in vertical stress & cylindrical cavity pressure (Ticino sand, Constant stress boundary, Normally consolidated, $D_r = 90\%$, applied vertical stress of 100 kPa).
CHAPTER 8

INTERPRETATION OF CONE PENETRATION TEST RESULTS IN UNSATURATED SANDS

8.1 INTRODUCTION

This chapter addresses the interpretation of the cone penetration test (CPT) in unsaturated sands. A comprehensive study of suction influences on CPT results is presented. In particular, it is attempted to quantify the contribution of suction to cone penetration resistance in relation to confining stress and initial state of sands. In chapter 7 the laboratory controlled CPT results conducted in the calibration chamber were corrected to infer what would be measured in free field conditions. Those corrected results are represented here along with CPT data reported in the literature from two other investigations in unsaturated sands and discussions are given on how the variations in the degree of saturation affect cone penetration resistance.

The concept of effective stress in unsaturated soils is used to develop a new method for interpretation of CPT results in unsaturated sands. Procedures are proposed for i) obtaining equivalent saturated CPT profiles from unsaturated CPT profiles where the CPT data for the saturated condition is not available, provided that the contribution of
suction ($\chi_s$) to the effective stress is known, and ii) obtaining the contribution of suction to the effective stress when CPT profiles for both unsaturated conditions and saturated (or dry) conditions are known. The procedures enable utilization of existing CPT correlations (developed for saturated/dry soils) for characterization of unsaturated soils.

Finally, established CPT correlations for dry/saturated sands are modified and extended for application to unsaturated sands. The new correlations enable direct characterization of unsaturated sands from CPT results provided that the contribution of suction to the effective stress ($\chi_s$) is known. This has been possible due to the similar influences constant suction and constant moisture content drainage conditions have on cavity expansion and CPT results, stemming primarily from an absence of suction hardening, the uniformity of $\chi_s$ for suction values commonly encountered, and the fact that SWCC is very flat for these suction values.

### 8.2 RELATIONSHIPS BETWEEN MEASURED, TOTAL AND EFFECTIVE CONE PENETRATION RESISTANCES

The corrected total cone resistance, $q_T$, is related to measured cone resistance, $q_c$, according to $q_T = q_c + u_n(1-a)$ for saturated conditions (as discussed in section 2.3) and $q_T = q_c - \chi_s(1-a)$ for unsaturated conditions, in which $a$ represents the cone area ratio and is approximately equal to the ratio of the cross-sectional area of the load cell shaft, $A_n$, divided by the projected area of the cone, $A_c$, as shown in Figure 2.5. For unsaturated condition suction acts on the shoulder area behind the cone and on the ends of the friction sleeve. $\chi_s$ is the contribution of suction to the effective stress. Standard cone penetrometers usually have an $a$ value within the range of 0.55 to 0.9 although in some standard sized cones this ratio maybe as low as 0.38 (Lunne et al., 1997). For the miniature cone used in this study $a = 0.11$, determined using the geometrical configuration of the cone and load cell.
For saturated sands, drained conditions prevail during cone penetration testing meaning no excess pore pressures are generated at the cone tip during penetration. As in-situ hydrostatic pressures are generally very small compared to the measured cone penetration resistances, meaning \( q_e \gg |\mu| \), it follows that \( q_e \approx q_T \). For this reason it is customary to interpret CPT data using \( q_e \) instead of \( q_T \) (Lunne et al., 1997), even in terms of effective stresses present in the sand. This custom will be adopted here, also for penetrations in unsaturated sands, as \( q_e \gg |\sigma_s| \).

### 8.3 COLLATION OF DATA AND GENERAL TRENDS

The corrected CPT results from chapter 7 are those which would be measured in free field conditions and are used here to investigate the CPT in unsaturated sands. They are summarized in Tables 8.1 and 8.2 along with corresponding suction and mean net stress values for specimens having relative densities of \( D_r = 33\% \) and 61\%, respectively.

Figures 8.1 and 8.2 highlight the influence of suction on cone penetration resistance for initial relative densities of \( D_r = 33\% \) and 61\%, respectively. It is observed that for a given mean net stress, cone penetration resistance increases as suction increases. It is also interesting to note that the contribution of suction to cone penetration resistance, represented by the horizontal shift of the data, is almost identical for various confining stress values.

Lehane et al. (2004) presented a series of CPT results for a site comprising Perth sand, containing less than 5\% fines and of relative density \( D_r = 45 \pm 10\% \). The tests were performed at different times corresponding to the end of a wet season and end of a dry season. Some tests were performed on parts of the site near large trees, while other tests were performed in an open area. Figures 8.3 and 8.4 summarize the results of tests conducted in the treed area and open area, respectively. Near the treed area it was found that, at the end of the dry season, \( q_c \) was significantly higher than the corresponding values at the end of the wet season. Also, in the open area, the seasonal
change had a very minor influence, if any at all, on the test results. The main conclusion drawn in the Lehane et al. (2004) investigation was that when $S_r$ is less than about 0.1, suction is large enough to have a significant effect on $q_c$, although the presence of tree roots was required to cause $S_r$ to drop below 0.1 and therefore increase suction significantly.

Table 8.1 Mean net stress, suction, mean effective stress and corrected cone resistance values for the CPTs conducted in saturated and unsaturated Sydney sand with an initial relative density of $D_r = 33\%$.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Mean net stress (kPa)</th>
<th>Suction (kPa)</th>
<th>Mean effective stress (kPa)</th>
<th>Corrected $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SL25</td>
<td>25.0</td>
<td>0</td>
<td>25</td>
<td>2.09</td>
</tr>
<tr>
<td>UL25-7-13</td>
<td>25.0</td>
<td>10</td>
<td>33.22</td>
<td>2.85</td>
</tr>
<tr>
<td>UL25-50</td>
<td>25.0</td>
<td>50</td>
<td>41.96</td>
<td>3.63</td>
</tr>
<tr>
<td>UL25-200</td>
<td>25.0</td>
<td>200</td>
<td>54.80</td>
<td>4.08</td>
</tr>
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<td>SL50</td>
<td>50.0</td>
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<td>3.53</td>
</tr>
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<td>SL50R</td>
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<td>0</td>
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<td>3.42</td>
</tr>
<tr>
<td>UL50-25</td>
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<td>200</td>
<td>79.80</td>
<td>5.35</td>
</tr>
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<td>200</td>
<td>79.80</td>
<td>5.56</td>
</tr>
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<td>0</td>
<td>100</td>
<td>5.97</td>
</tr>
<tr>
<td>UL100-25</td>
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<td>112.41</td>
<td>6.80</td>
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<td>112.41</td>
<td>6.70</td>
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<tr>
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<td>100.0</td>
<td>200</td>
<td>129.80</td>
<td>7.86</td>
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</table>
Table 8.2 Mean net stress, suction, mean effective stress and corrected cone resistance values for the CPTs conducted in saturated and unsaturated Sydney sand with an initial relative density of $D_r = 61\%$.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Mean net stress (kPa)</th>
<th>Suction (kPa)</th>
<th>Mean effective stress (kPa)</th>
<th>Corrected $q_c$ (MPa)</th>
</tr>
</thead>
<tbody>
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<td>0</td>
<td>30</td>
<td>3.30</td>
</tr>
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<td>25</td>
<td>42.41</td>
<td>4.19</td>
</tr>
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<td>59.80</td>
<td>5.21</td>
</tr>
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<td>112.41</td>
<td>12.90</td>
</tr>
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<td>112.41</td>
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<td>UM100-200</td>
<td>100</td>
<td>200</td>
<td>129.80</td>
<td>14.57</td>
</tr>
<tr>
<td>SM150</td>
<td>150</td>
<td>0</td>
<td>150</td>
<td>18.93</td>
</tr>
<tr>
<td>UM150-200</td>
<td>150</td>
<td>20</td>
<td>179.80</td>
<td>21.33</td>
</tr>
</tbody>
</table>

Similar observations in regard to influence of suction on cone penetration resistance were reported by Hryciw & Dowding (1987). They conducted an experimental investigation in Ottawa (quartz) sand although it is important to note that a much smaller and less sophisticated chamber was used than that used in present study. Specimens were prepared to a relative density of $D_r = 50\%$ and a range of degrees of saturation ($S_r$). The specimens in the chamber were not subjected to external confining
stresses during the tests, rather, a zero displacement boundary condition was used. Cone penetration resistance at 0.3 m depth was measured (where $p_{n0} \approx 5$ kPa corresponding to a unit weight of about 16 kN/m$^3$).

Figure 8.5 shows a plot of the Hryciw and Dowding (1987) data, without any corrections made to account for boundary influences, along with those generated here for Sydney sand. The cone resistances have been normalized against saturated values and plotted against degree of saturation, $S_r$. The normalized quantity is denoted by $q_c/q_{c,S_r=1}$ where $q_c$ is the cone resistance measured in an unsaturated test and $q_{c,S_r=1}$ is the cone resistance measured in a fully saturated specimen.

For the data generated in this study $S_r$ was calculated according to $S_r = wG_s/e$ where $w$, $G_s$, and $e$ denote specimen moisture content, specific gravity and void ratio respectively. A specific gravity of $G_s = 2.65$ was determined for Sydney sand and void ratios of $e = 0.813$ and 0.723 corresponds to relative densities of $D_r = 33\%$ and 61\% respectively (section 4.2). The moisture content values for each test were determined through post test core sampling as described in section 4.3.6. Note that there have been a good agreement between post test measured moisture contents and those obtained from soil water characteristic curves (Appendices C and D) for the corresponding values of applied suction. The same trend would be expected for the results if they are plotted against suction. The suction contribution to cone resistance strongly depends on mean net stress and when the results are normalised against their saturated equivalent values they do not show a collapse onto a single line.

The results of Hryciw and Dowding (1987) indicate that for $S_r$ values larger than about 0.65, suction induced in the specimens had negligible effect on the cone resistance. However, for $S_r$ less than about 0.1 there was a significant increase in cone penetration resistance. The results for Sydney sand indicate that suction induced in the specimens causes $q_c/q_{c,S_r=1}$ to rise above unity. The observed trend for Sydney sand is in close agreement with that of Hryciw and Dowding (1987) where at relatively low degrees of saturation ($S_r = 9\% - 15\%$), the normalized quantity ($q_c/q_{c,S_r=1}$) attains its maximum. However, the maximum value of the normalized quantity depends significantly on the
value of net confining stress. It can be observed that the normalized quantities generally increase as net confining stress decreases. A most significant influence of suction is observed for the tests conducted by Hryciw and Dowding (1987) where the specimens were subjected to a very low (5 kPa) confining stress.

Figure 8.6 represents contribution of suction to cavity pressure and cone penetration resistance in relation to mean net stress, where cavity pressure, $\sigma'_c$ (or cone resistance, $q_c$) is normalized against the value in saturated drained conditions denoted by $\sigma'_{c,s}$ (or $q_{c,s}$) corresponding to equal values of $p_{n0}$ and $D_r$. The normalized quantity is plotted against mean net stress ($p_{n0}$) for various initial suction values and initial relative density of $D_r = 33\%$ and $61\%$. The continuous lines represent cavity expansion results and corresponding suction values are indicated for each set of results. Unsaturated CPT results are represented by hollow symbols of different shape for any suction value. It can be seen that increasing suction, increases the normalized quantity and this becomes more significant for smaller values of net confining stress.

### 8.4 JUSTIFICATION OF THE CONSTANT SUCTION ASSUMPTION

Russell and Khalili (2006b) showed that for Sydney sand the soil response around expanding cavities under constant suction (drained) and constant moisture content (undrained) conditions were virtually indistinguishable for a given set of commonly encountered initial conditions. The indistinguishable behaviour was due to interaction of Equations (5.32) and (5.36), the absence of suction hardening, and the fact that SWCC is very flat for suction values encountered in practice. In other words small change in moisture content value takes place with significant increase or reduction of suction.

As cone penetration resistance is analogous to the cavity pressure (Bishop et al., 1945), the finding of Russell and Khalili (2006b) enables interpretation of measured cone penetration resistances in unsaturated sands assuming drained conditions prevail, even though a penetration rate of 20 mm/s was used, the soil moisture content around the cone tip remains constant. The assumption of constant suction, with an initial value
equal to the far field value, greatly simplifies interpretation of the CPT in unsaturated sands.

Note that for other soils, where suction hardening occurs and $\chi_s$ does not attain a constant value, the responses of constant suction and constant moisture content would not be identical and the constant suction assumption for unsaturated tests conducted at a penetration rate of 20 mm/s is unlikely to be valid.

8.5 A METHOD FOR INTERPRETING CPT RESULTS IN UNSATURATED SANDS

Recall that, following the work of Bishop (1959), the mean effective stress is defined as:

\[ p' = p_n + \chi s \]  \hspace{1cm} (8.1)

and that Russell and Khalili (2006a) obtained a relationship for the effective stress parameter ($\chi$) in terms of the suction ratio defined by (5.7):

\[
\chi = \begin{cases} 
1 & \text{for } \frac{s}{s_e} \leq 1 \\
(\frac{s}{s_e})^{-0.55} & \text{for } 1 < \frac{s}{s_e} \leq 25 \\
25^{0.45}(\frac{s}{s_e})^{-1} & \text{for } \frac{s}{s_e} > 25 
\end{cases}
\] \hspace{1cm} (8.2)

where $s_e$ is the suction value representing transition between saturated and unsaturated states.

Equations (8.1) and (8.2) permit the initial mean effective stress values to be determined for the soil in which each CPT was conducted in the calibration chamber.
and are tabulated in Tables 8.1 and 8.2 for specimens having relative densities of 33% and 61%, respectively. A plot of corrected cone resistance values versus initial mean effective stress is presented in Figure 8.7. The saturated test results are represented by solid symbols whereas unsaturated test results are presented by hollow symbols of the same shape for a particular relative density. Triangular symbols represent tests conducted in specimens having initial relative density of $D_r = 33\%$ and circular symbols represent tests conducted in specimens with initial relative density of $D_r = 61\%$.

Similar to the correlation proposed by Baldi et al. (1986) for Ticino sand, and following the discussion in section 8.2, cone penetration resistances ($q_c$) measured in Sydney sand through this study may be related to the mean effective stress ($p'_0$) and relative density according to (Figure 8.7):

$$q_c = 45(p'_0)^{0.85} \exp(2.78D_r)$$

(8.3)

Correlations similar to Equation (8.3) but using slightly different power exponents on $p'_0$ have been proposed for other sands (Jamiolkowski et al., 2001) and in the earlier studies vertical effective stress has been used in place of mean effective stress (e.g. Schmertmann, 1978; Villet and Mitchell, 1981). However, as pointed out by Baldi et al. (1986) and Jamiolkowski et al. (2001), a correlation using vertical effective stress is only valid for normally consolidated sands and correlations based on mean effective stress apply more widely to both normally and over-consolidated sands. Only Houlsby and Hitchman (1988) have used a power law in terms of horizontal effective stress.

Robertson and Campanella (1983a) carried out a review of the available calibration chamber test results and showed that the correlations between cone resistance and effective stress for a given relative density were similar in trend but were strongly influenced by sand compressibility (Figure 2.6). Therefore different power laws may apply to sands with various compressibility values as is evident through the range of correlations proposed in the literature. For example the correlations proposed by Schmertmann (1976) represent the results of tests performed on Hilton Mines sand,
which is a highly compressible quartz, feldspar, mica mixture with angular grains. The curves suggested by Villett and Mitchell (1981) represent results of tests conducted on Monterey sand with relatively low compressibility. Ticino sand used by Baldi et al. (1981, 1986) was a quartz, feldspar, mica mixture with subangular particles and appears to have a moderate compressibility. Generally sands with angular grains tend to be more compressible than sands having rounded grains.

Interestingly, a relationship similar to Equation (8.3) with \( q_c \) replaced by \( \sigma'_c \) and a slightly different power exponent (equal to 0.7) on \( p'_c \) can be used to fit the spherical cavity expansion results for Sydney sand (presented in Figure 6.2).

The advantage of having the type of relationship in Equation (8.3) is that the following equation applies for a given relative density:

\[
\frac{q_{c2}}{q_{c1}} = \left( \frac{p'_{c2}}{p'_{c1}} \right)^{0.85}
\]

(8.4)

It follows that if a CPT conducted in a sand when saturated or dry (such that \( p'_0 = p_{n0} - u_w \), where \( u_w \) is the pore water pressure and is zero when the sand is dry) and again in the same sand when unsaturated (such that \( p'_0 = p_{n0} + \chi S \)), the ratio between the cone resistance for unsaturated and saturated states, denoted using \( q_{c2} \) and \( q_{c1} \) respectively, is:

\[
\frac{q_{c2}}{q_{c1}} = \left( \frac{p_{n0} + \chi S}{p_{n0} - u_w} \right)^{0.85}
\]

(8.5)

Equation (8.5) was used to predict \( \chi S \) values for the CPTs conducted in the calibration chamber. More specifically, the corrected cone resistance values in Tables 8.1 and 8.2 were used for \( q_{c1} \) and \( q_{c2} \) as appropriate. The predicted \( \chi S \) values are tabulated in Tables 8.3 and 8.4 for initial relative densities of \( D_r = 33\% \) and 61\%, respectively.

Alternatively, as the initial suction values were known for all the tests conducted in the
calibration chamber, Equation 8.2 can be applied to obtain the $\chi_s$ values that were imposed in the experiments for comparison. An air entry value of $s_e = 7$ kPa was used in the calculations for all specimens.

This method for predicting the $\chi_s$ value in unsaturated Sydney sand gives a broadly similar result to the $\chi_s$ value imposed in the chamber. The power law of Equation (8.5) seems to be an appropriate and simple tool for back calculation of $\chi_s$, with errors being less than 9 kPa.

**Table 8.3** Predicted and imposed values of $\chi_s$ for the CPTs conducted within the calibration chamber in specimens having initial relative density of $D_r = 33\%$.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Mean net stress (kPa)</th>
<th>Suction (kPa)</th>
<th>Saturated cone resistance (kPa)</th>
<th>Unsaturated cone resistance (MPa)</th>
<th>Predicted $\chi_s$ (kPa)</th>
<th>Imposed $\chi_s$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UL25-7-13</td>
<td>25.0</td>
<td>10</td>
<td>2.09</td>
<td>2.85</td>
<td>8.22</td>
<td>11.01</td>
</tr>
<tr>
<td>UL25-50</td>
<td>25.0</td>
<td>50</td>
<td>2.09</td>
<td>3.63</td>
<td>16.96</td>
<td>22.86</td>
</tr>
<tr>
<td>UL25-200</td>
<td>25.0</td>
<td>200</td>
<td>2.09</td>
<td>4.08</td>
<td>29.80</td>
<td>29.92</td>
</tr>
<tr>
<td>UL50-25</td>
<td>50.0</td>
<td>25</td>
<td>3.53</td>
<td>4.38</td>
<td>12.41</td>
<td>14.45</td>
</tr>
<tr>
<td>UL50-200</td>
<td>50.0</td>
<td>200</td>
<td>3.53</td>
<td>5.35</td>
<td>29.80</td>
<td>31.55</td>
</tr>
<tr>
<td>UL50-200R</td>
<td>50.0</td>
<td>200</td>
<td>3.53</td>
<td>5.56</td>
<td>29.80</td>
<td>35.33</td>
</tr>
<tr>
<td>UL100-25</td>
<td>100.0</td>
<td>25</td>
<td>5.97</td>
<td>6.80</td>
<td>12.41</td>
<td>16.55</td>
</tr>
<tr>
<td>UL100-25R</td>
<td>100</td>
<td>25</td>
<td>5.97</td>
<td>6.70</td>
<td>12.41</td>
<td>14.54</td>
</tr>
<tr>
<td>UL100-200</td>
<td>100.0</td>
<td>200</td>
<td>5.97</td>
<td>7.86</td>
<td>29.80</td>
<td>38.21</td>
</tr>
</tbody>
</table>

Equation 8.5 may also be used to convert cone penetration resistance measured in an unsaturated sand to a corresponding penetration resistance for when the sand is
saturated/dry and subjected to the same externally applied net stresses and has the same relative density as long as the variation of $\chi_s$ with depth is known (assuming of course that the $u_w$ profile for saturated conditions is known). This enables the utilization of existing CPT correlations (developed for saturated/dry conditions) for characterization of unsaturated soils where CPT profiles in saturated/dry conditions are not available.

**Table 8.4** Predicted and imposed values of $\chi_s$ for the CPTs conducted within the calibration chamber in specimens having initial relative density of $D_r = 61\%$.

<table>
<thead>
<tr>
<th>Test number</th>
<th>Mean net stress (kPa)</th>
<th>Suction (kPa)</th>
<th>Saturated cone resistance (kPa)</th>
<th>Unsaturated cone resistance (MPa)</th>
<th>Predicted $\chi_s$ (kPa)</th>
<th>Imposed $\chi_s$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>UM30-25</td>
<td>30</td>
<td>10</td>
<td>3.30</td>
<td>4.19</td>
<td>12.41</td>
<td>9.73</td>
</tr>
<tr>
<td>UM30-200</td>
<td>30</td>
<td>200</td>
<td>3.30</td>
<td>5.21</td>
<td>29.80</td>
<td>21.34</td>
</tr>
<tr>
<td>UM50-25</td>
<td>50</td>
<td>25</td>
<td>5.29</td>
<td>6.17</td>
<td>12.41</td>
<td>9.92</td>
</tr>
<tr>
<td>UM50-200</td>
<td>50</td>
<td>200</td>
<td>5.29</td>
<td>7.17</td>
<td>29.80</td>
<td>21.51</td>
</tr>
<tr>
<td>UM100-25</td>
<td>100</td>
<td>25</td>
<td>11.59</td>
<td>12.90</td>
<td>12.41</td>
<td>13.43</td>
</tr>
<tr>
<td>UM100-25R</td>
<td>100</td>
<td>25</td>
<td>11.59</td>
<td>12.67</td>
<td>12.41</td>
<td>11.05</td>
</tr>
<tr>
<td>UM100-200</td>
<td>100</td>
<td>200</td>
<td>11.59</td>
<td>14.57</td>
<td>29.80</td>
<td>30.89</td>
</tr>
<tr>
<td>UM150-200</td>
<td>150</td>
<td>200</td>
<td>18.93</td>
<td>21.33</td>
<td>29.80</td>
<td>22.62</td>
</tr>
</tbody>
</table>

Furthermore, if CPT profiles at a particular site are available for saturated/dry and unsaturated conditions then the $\chi_s$ profile can be back-calculated for the unsaturated condition. Equation (8.5) will now be used to back calculate $\chi_s$ values from the data of Lehane et al. (2004) (Figure 8.3). At the end of the wet season it is assumed that hydrostatic pore water pressures are not present ($u_w = 0$) and that any effects of suction
are negligible. It follows that mean net and mean effective stresses are equal at the end of the wet season and may be determined using \( p_0 = p_0 = (1 + 2K_0)\gamma z/3 \), where depth is \( z \), and the horizontal earth pressure coefficient \( (K_0) \) and bulk density \( (\gamma) \) take values of 0.5 and 16.7 kN/m\(^3\) (as reported in Lehane et al., 2004). The determined values of \( \chi_s \) are plotted with depth in Figure 8.8. \( \chi_s \) has an upper limit of 69 kPa at 3.7 m depth.

8.6 EXTENSION OF ESTABLISHED CPT CORRELATIONS TO ACCOUNT FOR SUCTION EFFECTS

In this section, established correlations for interpretation of the CPT in sands (developed for saturated/dry conditions) are modified to account for influences of suction. The effective stress concept (as presented in section 8.5) is utilized in the development of the new correlations. The indistinguishable response of sands under constant suction (drained) and constant moisture content (undrained) conditions is assumed to apply, as discussed earlier. It is further assumed that the phenomenon of suction hardening does not occur in the considered sand types. The existing layout of the correlations has been maintained for convenience and to maintain familiarity for users.

8.6.1 Correlations for estimation of relative density

Relative density \( (D_r) \) is extensively used in geotechnical engineering as an indicator of the mechanical properties of sands. A number of empirical correlations, mainly based on calibration chamber investigations, have been developed enabling estimation of relative density from the CPT results. These correlations are still commonly used by practicing engineers and here it is attempted to extend the most commonly used ones to account for influences of suction. It is acknowledged that the compressibility of sands heavily influences the cone resistance - relative density correlations (Robertson, 2009) and for using this type of correlation, particular consideration should be given to the compressibility of the sand in which the CPT is performed.

Figures 8.9-8.11 represent correlations of cone resistance, vertical net stress and relative density for Hilton Mines sand (Schmertmann, 1976), Ticino sand (Baldi et al.,
1982) and Monterey sand (Villet and Mitchell, 1981) respectively. The correlations are extended to unsaturated conditions incorporating a range of $\chi_s$ values. The $\chi_s$ value for Sydney sand (determined using Equation (8.2)) increases by suction and approaches an asymptotic value once a certain suction value had been exceeded (equal to $25\sigma_{ce} = 175$ kPa). A maximum value of $\chi_s = 29.8$ kPa is attained using Equation (8.2) for Sydney sand but higher values of $\chi_s$ may be encountered for other sands, for example as is show in Figure 8.8 for Perth sand (Lehane et al., 2004). Therefore, $\chi_s$ values ranging from 12.5 kPa to 100 kPa have been considered in extension of the correlations to unsaturated sands to cover this possibility. The vertical axis was modified to be vertical net stress (the original correlations were presented using vertical effective stress). For a saturated test, and $u_w = 0$, mean net stress would be equal to mean effective stress and the original correlations are retrieved as shown by the continuous lines.

Figure 8.12 presents modified correlations of cone resistance, mean net stress and relative density for Ticino sand (Baldi et al., 1986) having a relative density ($D_r$) ranging from 20% to 100%. The modification included changing the vertical axis from mean effective stress to mean net stress. For a saturated sand, and $u_w = 0$, mean net stress is equal to mean effective stress. Extending the ideas of Baldi et al. (1986), the relative density correlations based on mean net stress are suited to both normally and overconsolidated sands while the correlations based on vertical stress are only valid for normally consolidated sands.

Figure 8.12 encompasses a broad range of relative densities and the differences between saturated and unsaturated conditions seem to be less significant for looser specimens. However, this apparent feature is due to the horizontal scale and in actual fact the difference between correlations of saturated and unsaturated states is also significant for loose sands as shown in Figure 8.13.

The proposed correlations in Figures 8.9-8.13 enable determination of relative density of an unsaturated sand from CPT results provided that the $\chi_s$ profile is known. A $\chi_s$ value can be calculated according to moisture content of the sand and its SWCC. It is observed that the relative density correlations are very sensitive to $\chi_s$. 

8-14
Moreover, Figures 8.9-8.13 highlight that failure to account for suction effects in interpreting CPTs in unsaturated sands can result in significant error in estimation of relative density. This is particularly crucial for lower values of mean net stress or vertical net stress which correspond to shallow penetrations. For example, suppose a CPT record in unsaturated sand (with a known value of $\chi_s = 70$ kPa) indicates cone penetration resistance equal to 7 MPa for a mean net stress of 50 kPa. Ignoring suction influence, a value of $D_r = 60\%$ is obtained using Figure 8.13 while incorporation of a $\chi_s$ value of 70 kPa leads to a value of $D_r = 40\%$ being obtained. Overestimation of relative density may lead to overestimation of strength and stiffness and this example clearly shows how neglecting suction influences can lead to an unconservative interpretation of unsaturated sand properties.

8.6.2 Correlations for estimation of drained shear strength of sand

The shear strength of sands is usually expressed in terms of the peak secant friction angle ($\phi'$). Robertson and Campanella (1983a) proposed correlations between $\phi'$, vertical effective stress and cone penetration resistance based on an extensive review of calibration chamber test results and measured friction angle values from drained triaxial tests. The correlations are represented in Figure 8.14 and were extended to account for influences of suction in unsaturated sands. The correlations shown in Figure 8.14 provide reasonable estimates of friction angle for normally consolidated, moderately compressible, predominantly quartz sands. For highly compressible sands, the chart would tend to predict conservatively low friction angles (Robertson and Campanella, 1983a).

In order to extend the friction angle correlations, $\chi_s$ values ranging from 12.5 to 100 kPa have again been considered. The vertical axis was modified to be vertical net stress rather than vertical effective stress and for a saturated condition with $u_w = 0$ the original correlations are retrieved and represented with continuous lines. It is observed that the correlations are notably altered for unsaturated sands and the shift between saturated and unsaturated correlations increases as the value of $\chi_s$ increases. The requirement of considering suction influences for evaluation of friction angle becomes crucial for lower vertical stresses when unsaturated correlations overlap those for...
saturated correlations. The low stress – low cone resistance portion of the chart has been magnified in Figure 8.15.

Now an example is used to highlight the resulting error associated with neglecting suction influences. Suppose cone resistance of 4 MPa was obtained for a certain depth from a CPT conducted in unsaturated sand where the vertical net stress was 75 kPa. It is further assumed that a value of $\chi_s = 70$ kPa applies, evaluated using the ground moisture content and its relation to suction through the soil-water characteristic curve. If suction influences are neglected a value of $q_c$ would be determined from Figure 8.14, compared to a value of $\phi' = 34^\circ$ if the influences of suction are appropriately considered. It can be seen that failure to account for suction effects can easily lead to an (often unconservative) error of 4° in the estimated $\phi'$ values.

### 8.6.3 Correlations for estimation of state parameter

An approach proposed by Been and Jefferies (1985) uses the state parameter as an alternative to relative density for characterization of sands. The state parameter (defined in section 5.3.6) is a quantitative measure of the state of a sand that combines the effect of void ratio and stress level with reference to the critical state, and can be used to describe a range of sand properties including peak friction angle.

Based on a study of calibration chamber tests, Been et al. (1986, 1987) developed correlations for estimation of the state parameter from CPT results. The correlations were presented in terms of normalized cone resistance $(q_c - p_n)/p'$ against the state parameter (denoted here using $\zeta$). The correlations of Been et al. (1987) for Monterey, Ticino, Hokksund, Ottawa, Reid Bedford and Hilton Mines sands have been extended in Figure 8.16 to account for influences of suction. The vertical axis was modified to a normalized quantity of $(q_c - p_n)/p_n$ and values of $(p_n + \chi_s)/p_n$ ranging from 1 to 5 were incorporated into the correlations to account for differing influences of suction. A value of $(p_n + \chi_s)/p_n = 1$ retrieves the original correlations for saturated sands for which the normalized quantity defining the vertical axis becomes $(q_c - p_n)/p'$. 
The correlations in Figure 8.16 can be used in conjunction with the relationship shown in Figure 8.17 for estimation of peak friction angle ($\phi'$) as suggested by Been and Jefferies (1985). It is important to note that the state parameter determined using this approach inherently incorporates the mean effective stress of the sand in the unsaturated state. The peak friction angle extracted from Figure 8.17 is therefore also relevant to when the sand has a mean effective stress equal to that in its unsaturated state.

Now an example is used to describe how the new correlations can be applied. The resulting errors associated with neglecting suction influences are highlighted through the example. Suppose a cone resistance of 4 MPa was obtained for a certain depth from a CPT conducted in unsaturated Hilton Mines sand where the mean net stress was 25 kPa. It is further assumed that a value of $\chi_s = 50$ kPa applies. Values of $q_c = 4$ MPa, $p_n = 25$ kPa and $\chi_s = 50$ kPa correspond to $(q_c - p_n)/p_n = 159$ and $(p_n + \chi_s)/p_n = 3$. If suction influences are neglected and $(p_n + \chi_s)/p_n = 1$ is assumed, a value of $\zeta = -0.23$ would be determined from Figure 8.16 (f), compared to a value of $\zeta = -0.11$ if the influences of suction are appropriately considered using $(p_n + \chi_s)/p_n =3$. Referring to Figure 8.17 reveals that failure to account for suction influences would result in an overestimation of friction angle by about 5 degrees.

8.7 CONCLUDING REMARKS

Results of CPTs in unsaturated Sydney sand inferred from the calibration chamber investigations and two studies of the CPT in unsaturated sands reported in the literature have been discussed. Influences of suction on cone penetration resistance have been investigated and it is observed that suction noticeably influences CPT results. The influences are more significant for lower confining stresses which correspond to shallow penetrations. A study of cone penetration resistance against degree of saturation ($S_r$) reveals that when $S_r$ is less than about 0.1, suction is large enough to have a significant effect on CPT results.
A simple method is presented for prediction and interpretation of the suction influences on CPT results. The method may be used to convert cone penetration resistance measured in an unsaturated sand to a corresponding penetration resistance for when the sand is saturated/dry as long as the variation of $\chi_s$ with depth is known. This enables utilization of existing CPT correlations (developed for saturated/dry conditions) for characterization of unsaturated soils where CPT profiles in saturated/dry conditions are not available. Furthermore, if CPT profiles at a particular site are available for saturated/dry and unsaturated conditions then the $\chi_s$ profile can be back-calculated for the unsaturated condition.

Also, established correlations for interpretation of the CPT in sands (developed for saturated/dry conditions) are extended to account for differing influences of suction. The new correlations enable direct characterization of an unsaturated sand from CPT results provided the contribution of suction to effective stress ($\chi_s$) is known. Commonly used correlations for estimation of relative density, friction angle and state parameter are considered and discussions are given on the importance of considering suction influences while interpreting CPTs in unsaturated sands. It is shown that neglecting suction influences while interpreting CPT results in unsaturated sands can lead to an unconservative estimation of engineering properties.
Figure 8.1 Influence of suction on cone resistance ($q_c$) for specimens with initial relative density of $D_r = 33\%$ and subjected to different values of mean net stress.
Figure 8.2 Influence of suction on cone resistance \( (q_c) \) for specimens with initial relative density of \( D_r = 61\% \) and subjected to different values of mean net stress.
Figure 8.3 In-situ cone penetration resistance for Perth sand in a treed area at the ends of wet season (represented by solid symbols) and dry season (represented by hollow symbols) (after Lehane et al., 2004).

Figure 8.4 In-situ cone penetration resistance for Perth sand in an open area at the ends of wet season (represented by solid symbols) and dry season (represented by hollow symbols) (after Lehane et al., 2004).
Figure 8.5 Normalized cone penetration resistances plotted against degree of saturation, for Sydney sand with initial relative densities of $D_r = 33\%$ and $61\%$ subjected to mean net stresses of 25 kPa, 30 kPa, 50 kPa, 100 kPa, and 150 kPa (represented by hollow symbols) along with test results for Ottawa sand (after Hryciw and Dowding, 1987) with initial relative density of $D_r = 50\%$ subjected to mean net stress of 5 kPa (represented by solid symbols).
Figure 8.6 CPT results for Sydney sand in the \( (q_c/q_{c,s}) \sim p_n^0 \) plane and cavity expansion results in the \( (\sigma'_c/\sigma'_{c,s}) \sim p_n^0 \) plane for specimens subjected to a range of suction values with initial relative density of a) \( D_r = 33\% \) and b) \( D_r = 61\% \).
Figure 8.7 Cone resistance ($q_c$) values plotted against mean effective stress ($p'_0$) values for the CPTs conducted in saturated and unsaturated Sydney sand with initial relative densities of 33% and 61%.

$$D_r = \frac{1}{2.78} \ln \left[ \frac{q_c}{45(p'_0)^{0.85}} \right]$$
Figure 8.8 $\chi_s$ values back-calculated from Lehane et al. (2004) CPT data (in a treed area) using Equation 8.9.
Figure 8.9 Correlations of cone resistance, vertical net stress and relative density for Hilton Mines sand (high compressibility) to account for differing influences of suction, extended from Schmertmann (1976).
Figure 8.10 Correlations of cone resistance, vertical net stress and relative density for Ticino sand (moderate compressibility) to account for differing influences of suction, extended from Baldi et al. (1982).
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Figure 8.12 Correlations of cone resistance, mean net stress and relative density (for a range of $D_r = 20\%-100\%$) for Ticino sand to account for influences of suction, extended from Baldi et al. (1986).
Figure 8.13 Correlations of cone resistance, mean net stress and relative density (for a range of $D_r = 20\% - 60\%$) for Ticino sand to account for influences of suction, extended from Baldi et al. (1986).
Figure 8.14 Correlations of cone resistance, vertical net stress and friction angle to account for influences of suction, extended from Robertson and Campanella (1983a).
Figure 8.15 Magnified low stress - low cone resistance portion of the correlations presented in Figure 8.14.
Chapter 8  
Interpretation of CPT in unsaturated sands

(a)

(b)
Chapter 8
Interpretation of CPT in unsaturated sands

(c) Hokksund

(d) Ottawa
Figure 8.16 Correlations of normalized cone resistance and state parameter for a) Monterey, b) Ticino, c) Hokksund, d) Ottawa, e) Reid Bedford and f) Hilton Mines sands to account for influences of suction, extended from Been et al. (1987); Upper and lower limits of \((q_c-p_n)/p_n\) values represent the range of experimental data considered by Been et al. (1987).
Figure 8.17 Correlation between state parameter and peak friction angle of sand (after Been and Jefferies, 1985).
SUMMARY AND CONCLUSIONS

9.1 GENERAL

The main objective of this study was to advance experimental and theoretical bases for interpreting the cone penetration test (CPT) in unsaturated sands. A comprehensive program of equipment development, laboratory experimentation and theoretical investigation was completed to study the CPT in unsaturated sands. The main tasks accomplished include:

1) Design, construction and successful operation of a new calibration chamber suitable for conducting CPTs in unsaturated soils.

2) Establishment of an appropriate specimen formation technique and testing procedure for conducting laboratory controlled CPTs in unsaturated Sydney sand.
3) Completion of an extensive program of CPTs in dry, saturated and unsaturated Sydney sand and evaluation of suction influence on CPT results.


5) Investigation of chamber size and boundary influences on CPT results using the cavity expansion solution procedure and development of a method for correction of CPT results conducted in the calibration chamber to infer what would be measured in free field conditions.

6) Development of a new method for interpreting CPT results in unsaturated sands and extension of established CPT correlations for use in unsaturated sands.

9.2 A NEW CALIBRATION CHAMBER FOR UNSATURATED SOILS

A calibration chamber suitable for conducting cone penetration tests in dry, saturated and unsaturated soils has been developed. The chamber can accommodate cylindrical specimens with a height of 840 mm and diameter of 460 mm. The novel aspects of the design include specimen formation system, a modified axial load application system, and the use of an enhanced axis translation technique for application, measurement and control of suction within the system.

Lateral confining pressure is applied by water pressure acting on a durable rubber membrane enclosing the soil specimen. Vertical pressure is applied by a hydraulic loading ram pushing on the chamber piston connected to the base of the specimen. Eight high air entry value porous ceramic disks are embedded in the upper face of the bottom plate, which enables control and maintenance of suction in the soil specimen using the axis translation technique.
Incorporation of a moveable former in the design enables greater control during preparation of soil specimens within the chamber as it resolves problems associated with lateral bulging. The specimen former consists of four stainless steel cylinder quarters. Two handles are attached to each cylinder quarter near the top and bottom; this enables manual movement of the quarters towards and away from the center of the chamber. During specimen preparation, the quarters of the former are pushed together and locked into position to form a rigid cylindrical mould.

After specimen preparation, the chamber is assembled and a confining cell pressure is applied. The quarters of the former may then be pulled away from the specimen to allow a constant-stress condition at the specimen boundary. Specimens of different soil types (cohesive or granular) can be prepared using compaction or pluvial deposition techniques adding to the versatility of the chamber.

The calibration chamber is fitted with a number of control units to apply and maintain cell pressure, pore water pressure, pore air pressure and vertical stress. The measurement devices record volume changes of the cell water and pore water and displacement of the specimen base.

Cone penetration tests are conducted using a miniature electrical cone having a diameter of 16 mm. A loading frame was specially built to mount the penetrometer and positioned above the chamber prior to testing. A specifically designed hollow bush cylinder was used to create a seal around the cone and the centre hole of the chamber top cap during specimen consolidation and penetration.

The calibration chamber was manufactured, assembled and operated successfully and results of laboratory controlled CPTs conducted in dry, saturated and unsaturated sand within the calibration chamber have been presented.
9.3 EXPERIMENTAL PROCEDURES AND CPT RESULTS IN SAND

Numerous cone penetration tests have been conducted on saturated, dry and unsaturated Sydney sand within the calibration chamber to study the influence of suction on cone resistance.

Dry sand specimens were prepared in the calibration chamber using the pluvial deposition technique. A uniform flow rate of sand through the diffuser and a uniform drop height (distance between diffuser and placed sand surface) enabled preparation of homogenous specimens. A targeted relative density was reached by adjusting the sand flow rate and drop height. Decreasing the flow rate and increasing the drop height increased the density.

Specimen saturation was achieved by passing de-aired distilled water through the perforated copper tubes located at the bottom of the specimen while applying a 10 kPa vacuum to the top of the specimen. Unsaturated specimens were formed by first saturating the specimens and then letting the moisture content reduce to achieve a target suction. Matric suction was applied to the specimens by increasing the air pressure connected to the chamber top cap while maintaining constant pore water pressure applied at the specimen base through high air entry disks. It was observed that connecting laboratory air pressure directly to the top cap fittings resulted in local drying of the specimen surface which was caused by pore water evaporating into the dry air. Therefore, to prevent localized drying, the laboratory air was passed through a water bath cylinder before entering the chamber.

The CPTs were conducted on the specimens subjected to a range of relative densities, confining net stresses and initial controlled suctions. The main observations are summarized as follows:

- Suction noticeably increases the average cone resistance in sands and the effect of suction is less prominent for small suction values.
- Suction can increase cone resistance by as much as 90% for a net confining stress of 25 kPa.

- Suction can increase cone resistance by 50% and 34% for specimens subjected to net confining stress of 50 kPa and having initial relative densities of $D_r = 33\%$ and 61%, respectively.

- Suction can increase cone resistance by 25% to 31% for a net confining stress of 100 kPa.

- The contribution of suction to the cone resistance becomes more significant as the net confining stress decreases and thus the influence of suction is more important for shallow penetrations (up to 5 m) where soil is most likely to be unsaturated.

- The effect of suction on the cone resistance is more pronounced for loose specimens.

A total of 4 CPTs were repeated to demonstrate repeatability of test results and the specimen formation methods adopted. Also, the observed similarity between test results obtained from dry and saturated specimens with identical effective confining stresses adds to the reliability of the results.

Post test core samples taken from different locations of the chamber specimens demonstrated that an even distribution of moisture content had been achieved throughout the specimen. There is also a good agreement between post test measured moisture contents and those obtained from soil-water characteristic curves for the corresponding values of applied suction. The core sample obtained from middle of the chamber specimen indicated that some particle crushing occurred in the vicinity of the cone penetrated. However, more advanced methods of sampling would be required to quantify the extent of particle crushing and its variation with distance from the cone tip.
9.4 CONSTITUTIVE MODEL, CALIBRATION AND SIMULATIONS

A conventional elastic-plastic hardening/softening constitutive model formulated in a critical state framework has been used to describe the stress-strain behavior of saturated and unsaturated sands. The concept of effective stress in unsaturated soils has been incorporated in the definition of the basic model ingredients. Also a simple isotropic elastic rule was adopted along with a non-associative flow rule. The model takes into account hardening due to plastic volumetric strains as well as suction in unsaturated conditions.

The model was calibrated for saturated and unsaturated Sydney sand to provide a good match between simulation and triaxial test results for saturated and unsaturated conditions. The model predictions are in particularly good agreement with experimental results at large shear strains as the critical state is approached. This is an important feature as the model is implemented into a cavity expansion analysis as cavity expansion causes a soil to undergo large strains and the critical state is reached by the soil at the cavity wall.

9.5 DRAINED CAVITY EXPANSIONS IN SOILS OF FINITE RADIAL EXTENT

A new solution procedure has been developed for solving the cavity expansion problem in soils of finite radial extent, for the first time incorporating a hardening/softening elastic-plastic constitutive model formulated in the critical state framework.

The governing equations in the finite elastic region and elastic-plastic region have been described and a simplifying assumption was introduced when converting a variable from rate form to differential form permitting an analytically derived solution to be obtained. The errors associated with the simplification have been investigated and shown to be less than 10%, and usually less than 5%, for conditions of relevance.
The stress conditions, specific volume and radial velocity at the elastic-plastic boundary are used as the initial values to solve the system of governing differential equations in elastic-plastic region. Of particular interest is stresses at the cavity wall which can be related to the cone resistance of a CPT.

The results generated are relevant only at the instant when the cavity has expanded to a certain size corresponding to a dimensional configuration of the cone and the chamber. The results do not represent the stress history of the soil at the cavity wall during expansion, as they do when cavity expansion occurs in an infinite soil mass. For the case of an infinite medium the differential equations become identical to those that would be derived using the similarity technique.

Using the new procedure cavity expansion results were generated for a quartz sand of infinite and finite radial extent and when subjected to both constant stress and zero displacement boundary conditions. The results were presented in a number of planes to explore and highlight the influences of the boundary condition type and boundary extent.

In particular, the radial stress at the cavity wall \( (\sigma'_c) \) was normalized against its value for expansions in infinite media (having the same values of \( p'_0 \) and \( v_0 \) prior to expansion). For the constant stress boundary condition, and for a given value of \( p'_0 \), the normalized quantity \( (\sigma'_c/\sigma'_{c,\infty}) \) increases and approaches unity as \( B/c \) increases, where \( B \) and \( c \) denote the radii of the finite boundary and the cavity, respectively. For the zero displacement boundary condition, and for a given value of \( p'_0 \), the \( \sigma'_c/\sigma'_{c,\infty} \) value reduces and approaches unity as \( B/c \) increases. For both boundary conditions, the size effects are more pronounced for decreasing values of \( p'_0 \) and increasing values of \( D_r \).

For spherical cavity expansions and both boundary conditions, the boundary size effects are insignificant for values of \( B/c \) larger than about 35. For cylindrical cavity expansions and the constant stress boundary condition, the boundary size effects become insignificant for values of \( B/c \) larger than about 140, and for the zero
displacement boundary condition, even larger values of $B/c$ are required before the boundary size effects become insignificant.

The pressure at a spherical cavity wall is influenced less by the size of the finite soil than is the pressure at a cylindrical cavity wall. Also, the pressure at the cavity wall, for both cylindrical and spherical cavities, is influenced less by the finite size of the soil for the constant stress boundary condition than for the zero displacement boundary condition.

The solution procedure presented is relevant to both saturated and unsaturated sands when drained conditions prevail. Also, as undrained and drained cavity expansions in unsaturated sands cause virtually indistinguishable soil responses, and assuming cone penetration resistance is analogous to the pressure required to expand a cavity, the procedure can be applied directly to interpretation of boundary influences on cone penetration resistances measured in calibration chambers.

9.6 CALIBRATION CHAMBER BOUNDARY EFFECTS

The calibration chamber boundary and size effects on CPT results were investigated using the cavity expansion theory. It is shown for CPTs conducted in specimens subjected to constant stress boundary conditions within a calibration chamber that the size effects are related to both the reduction in vertical stress state behind the cone and the finite extent of the boundary. These two factors cause separate decreases in cone resistance as the diameter ratio decreases.

Predictions made by spherical cavity expansion analysis incorporating changes in vertical stress behind the penetrating cone are in good agreement with experimental results from the literature. Spherical cavity expansion analyses show that the finite extent of the specimen boundary causes influences that are more pronounced for expansions in dense sands.
The reduction in the vertical stress behind the penetrating cone and consequently the reduction in mean stress behind the penetrating cone tip is significant for high values of cone resistance which are inherent of tests conducted in dense specimens. The initial values of horizontal and vertical stress, and their relative magnitude also influence the severity of the observed boundary effects. Repeating the analyses using cylindrical cavities provides a very poor fit to the data, indicating that spherical cavity expansion analysis is the more suitable tool for interpretation. Size effects should be considered in the interpretation of the CPTs conducted in calibration chambers.

A new procedure makes it possible to correct the results to a free field condition with good precision. The technique has been used to infer the free field values of cone resistance for the CPT results conducted within the calibration chamber. The maximum predicted size effect is less than 8% for the tests performed in the calibration chamber.

9.7 INTERPRETATION OF CONE PENETRATION TEST RESULTS IN UNSATURATED SANDS

Results of CPTs in unsaturated Sydney sand from the calibration chamber have been studied along with those from two CPT investigations in unsaturated Perth and Ottawa sands reported in the literature. General trends of CPTs in unsaturated sands have been discussed.

In particular, differing influences of suction on cone penetration resistance have been investigated. It is observed that suction noticeably influences cone penetration resistance. The influences are more significant for lower confining stresses which correspond to shallow penetrations. A study of cone penetration resistance in relation to degree of saturation ($S_r$) in sands reveals that when $S_r$ is less than about 0.1, suction is large enough to have a significant effect on CPT results.

The close similarity between constant suction and constant moisture content cavity expansion results for unsaturated Sydney sand permits interpretation of cone
penetration test results assuming constant suction (drained) conditions prevail. Cone penetration resistance is analogous to the cavity pressure and the assumption of constant suction, with an initial value equal to the far field value, greatly simplifies interpretation of the CPT in unsaturated sands.

The method of interpretation may be used to convert cone penetration resistance measured in an unsaturated sand to a corresponding penetration resistance for when the sand is saturated/dry as long as the variation of $\chi_s$ with depth is known. This enables utilization of existing CPT correlations (developed for saturated/dry conditions) for characterization of unsaturated soils where CPT profiles in saturated/dry conditions are not available. Furthermore, if CPT profiles at a particular site are available for saturated/dry and unsaturated conditions then the $\chi_s$ profile can be back-calculated for the unsaturated condition.

Also, established correlations for interpretation of the CPT in sands (developed for saturated/dry conditions) have been extended to account for differing influences of suction. The extended correlations enable direct characterization of an unsaturated sand from CPT results provided the contribution of suction to effective stress ($\chi_s$) is known. The indistinguishable response of sands under constant suction (drained) and constant moisture content (undrained) conditions is assumed to apply. It is further assumed that suction hardening does not occur in the considered sand types.

Discussions are given on the importance of considering suction influence while interpreting CPTs in unsaturated sands. It is shown that neglecting suction influences while interpreting CPT results can lead to unconservative estimation of engineering properties.
9.8 SUGGESTIONS FOR FURTHER RESEARCH

Suggestions for future studies are as follows:

1) Study the influence of suction on CPT results in fine-grained soils within laboratory controlled conditions.

2) Develop a solution procedure for undrained cavity expansions in fine-grained soils of finite radial extent in saturated and unsaturated conditions.

3) Develop a method for interpreting CPT results in unsaturated fine-grained soils.

4) Perform a comprehensive experimental investigation of calibration chamber size effects on CPT results within various soil types.

5) Investigate the exact relationship between cavity limit pressure and CPT results, considering spherical, elliptical and possibly conical cavity expansions.


References


References


References


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References


References


References


Vesic, A.S. (1975) Principles of pile foundation design. Lecture Series on Deep Foundations Lecture 1, Boston Society of Civil Engineers Section, ASCE.


APPENDIX A

DRAWINGS OF THE DESIGNED CALIBRATION CHAMBER

The design and manufacturing of the calibration chamber was crucial to investigate the cone penetration tests in unsaturated sand. The detailed design drawings of the calibration chamber are presented here.

General arrangements of the calibration chamber and reference to details are presented in drawing 1. Drawing 2 details the joining of the flanges to the chamber shell. In other drawings main components of the calibration chamber are presented including: Moveable former (drawings 3-7), sample bottom plate (drawings 8-11), chamber wings and ring (drawings 12 -13), chamber piston and hydraulic ram set up (drawings 14-16), chamber valves and connections (drawings 17-18), arrangement of penetrometer and frame (drawings 18-19) and details of the bush cylinder (drawings 20-21). All the dimensions shown are in mm.
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Appendix A

Drawings of the calibration chamber

Drawing 1: General Arrangements of Calibration Chamber
Appendix A  Drawings of the calibration chamber

Drawing 2  Attachment of Flanges to Chamber Shell (Detail A & B)
Appendix A

Drawings of the calibration chamber

Drawing 3  General View of Moveable Former (Detail C)
Drawing 4  Plan View of Moveable Former (Section K-K)

Section K-K

Support rod
$D = 19 \text{ mm (3/4 in)}$
Two every $90^\circ$

Stainless steel flange
Split every $90^\circ$

Hexagonal-head bolt
$D = 12.7 \text{ mm (1/2 in)}$
Positioned every $45^\circ$
Appendix A

Drawings of the calibration chamber

Section L-L

Detail C-1

Stainless steel flange
ID: 486 mm
OD: 546 mm
Thickness: 10 mm

Hexagonal-head bolt
D = 12.7 mm (1/2 in)
Positioned every 45°

Split every 90°

Drawing 5 | Moveable Former (Sections L-L & Detail C-1)
Drawing 6
Moveable Former (Detail C-2)
Appendix A
Drawings of the calibration chamber

Drawing 7  Moveable Former  (Detail C-3)

Support pin hole
D = 19 mm (3/4 in)
Two every 90°
Appendix A

Drawings of the calibration chamber

Detail D

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To air flush

To pore water pressure control

Cavity to house a high air entry value porous disk
Positioned every 45°
D = 104.78 mm
N = 8

Curved groove in base of cavity
Diffused air collector
Width: 3 mm
Depth: 3 mm
Appendix A

Drawings of the calibration chamber

Detail D - Underside View

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D = 8.75 mm
D = 4 mm
D = 8.75 mm

to pore water pressure control
To bleeding valve
To centre of disks

Valve connection
Pore water pressure

163.5 mm
196.6 mm
Appendix A

Drawings of the calibration chamber

Section R - R

Section S - S

Drawing 10
Bottom Plate (Sections R-R & S-S)
Appendix A

Drawings of the calibration chamber

Detail D - 2

Epoxy resin

High air entry value porous disk
D = 104.78 mm

Region in which curved groove is located

Drawing 11
Bottom Plate (Detail D-2)
Detail H

Chamber ring
ID: 330 mm
OD: 850 mm
Thickness: 40 mm

Durable plastic sleeve

Chamber wing
Cadmium plated
Length: 259 mm
Thickness: 20 mm
Height: 290 mm
Positioned every 45°

Chamber base plate
Diameter: 850 mm
Thickness: 30 mm

10 mm

M

M

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Drawing 13  Chamber Wings (Section M-M)

Section M-M
Appendix A

Drawings of the calibration chamber

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Drawing 14  Chamber Piston (Detail F)

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Drawing 15

Chamber Piston (Sections P-P & Q-Q)
Appendix A

Drawings of the calibration chamber

A-18

| Drawing 16 | Hydraulic Ram Set up (Detail G) |

Detail G

Hydraulic Ram
(when fully extended)

Ram Base Plate
Thickness: 15 mm

Ram base
Appendix A

Drawings of the calibration chamber

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Drawing 17  General View of Chamber Connections
Detail I

Rubber O-Ring groove

To cell pressure control system
To pore water pressure control system
To pore air pressure control system

Detail E

Plastic tubes

To pore water pressure control system
To air bleeding valve
To cell pressure control

Drawing 18
Chamber Connections (Detail E & I)
Drawing 19  General View of Penetrometer, Bush Cylinder and Frame
Appendix A
Drawings of the calibration chamber

Detail T

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Appendix A

Drawings of the calibration chamber

Drawing 21  Bush Cylinder (Sections U-U & V-V)

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APPENDIX B

CALIBRATION OF THE PENETROMETER
### A: Tip Resistance:

- **Bridge feeding (current)**: 13.193 mA
- **Sensitivity**: 0.3790 mV/mA
- **Bridge feeding (voltage)**: 4.712 V
- **Sensitivity**: 1.0511 mV/V
- **Inaccuracy less then**: 0.05 kN
- **Nom. Tip Resistance**: 10 kN
- **Max. Tip Resistance**: 20 kN
- **Output at nominal Tip Resistance**: 5 mV
- **Effective area**: 2 cm²

### B: Tip and local friction:

- **Bridge feeding (current)**: 13.414 mA
- **Sensitivity**: 0.3727 mV/mA
- **Bridge feeding (voltage)**: 4.773 V
- **Sensitivity**: 1.0476 mV/V
- **Inaccuracy less then**: 0.05 kN
- **Nom. Tip and local friction**: 10 kN
- **Max. Tip and local friction**: 20 kN
- **Output at nominal Tip and local friction**: 5 mV
- **Effective area**: 30 cm²

### C: Pore Pressure:

- **Sensortype F**
- **Bridge feeding (current)**: 1.226 mA
- **Sensitivity**: 16.3106 mV/mA
- **Bridge feeding (voltage)**: 1.354 V
- **Sensitivity**: 14.7700 mV/V
- **Inaccuracy less then**: 200 kPa
- **Nom. Pore Pressure**: 20000 kPa
- **Max. Pore Pressure**: 30000 kPa
- **Output at nominal Pore Pressure**: 20 mV

Calibrated by: C.J. Ouwejan  
Signature:  
Heerenveen, 13/09/2007
[CONES]

<cone 1>
cone number=070916
cone type=SUBP200-2
calibration date=13/09/2007
screen nr=screen 1
comment=Tip 50 MPa Sleeve 0.75 MPa Pore 20MPa
<<TIP>>
cci init=F5E1237AD[13]
cci input=1
cci output=1
scaling factor=1.0994166667
auto zero=on
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exitation=13.193
sensor displacement=0
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scaling offset=0
auto zero=on
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maximum zero offset=5
exitation=13.414
sensor displacement=4
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cci init=F5E1237AD[13]
cci input=3
cci output=3
scaling factor=49.046
scaling offset=0
auto zero=on
zero offsets after testing=0
maximum zero offset=2000
exitation=1.2262
sensor displacement=2
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1 Data structure GORILLA!

1.1 GENERAL
GORILLA!® is a multifunctional software program with several applications. Thanks to its "open" structure, the software (as well as its updates as they become available) is easy to operate. It is also easy for the user to adapt the software to his own needs.

Data is stored in ASCII-format, as ASCII files are universally applicable. The files can be installed as part of practically all data processing programs (databases, spreadsheets, word processing programs). Files in ASCII format take more space than files in binary format. In order to keep the files as small as possible, we chose for the structure below. Apart from that, ASCII files are easy to compress.

Data is stored under a filename. This filename consists of the job number and the test number - easy to recognise, sort and filter. The name of the file is the job number with the test number as its extension:

\[ \text{e.g.} \quad \text{job number} = 9578012A \\
\text{test number} = 15 \\
\text{filename} = 9578012A.015 \]

1.2 TEST DATA ORGANISATION
Test data is organised into groups, parts and items for an easy operation. Please read the example below to understand the structure.

All data belonging together will be stored in a group. The beginning of groups is indicated by text in []. Within a group are one or more parts, which are indicated by text in < >. Items, containing general information, consist of 1 or more lines. An item is followed by the sign =, which in its turn is followed by the information belonging to it. You store information using the order below:

1. General information
2. Calibration information and zero values of the parameters before the test is started
3. Test data
4. Zero values of the parameters after the test

Example:
(TEST)
<GORILLA! header>
job number=9978012A
client=ABF
job description=Deer Reecenveen Zuid
elevation=6

test number=019

date=34-3-1995
time=17:46
cone type=ELC10CFP16
cone serie =910507
inclination type=INCL 16/16
inclination serie =921156R
operator=PVA
reference level=0.0
reference point=PZF
GORILLA! remark 1=
GORILLA! remark 2=
stock #=711207634
MS version=MS TSB 4.6
MS serie =MS-03
CCI version=CCI 4.6
CCI serie=#
GORILLA! version=TSH ABB 6.2
GORILLA! serie=#9405.01

(SCALING FACTORS)
<TIP>= 1.000
<LOCAL FRICTION>= 1.000
<PORE PRESSURE>= 1.000

(SCALING OFFSETS)
<TIP>= 0.000
<LOCAL FRICTION>= 0.000
<PORE PRESSURE>= 0.000

(ZERO OFFSETS BEFORE TESTING)
<TIP>= 0.530
<LOCAL FRICTION>= 0.006
<PORE PRESSURE>= 1.000

(DATA)
* D: 0.02h1:0.0h2:0.0h3:1.0
* D: 0.04h1:0.0h2:0.0h3:1.0
* D: 0.06h1:0.0h2:0.0h3:1.0
* D: 0.08h1:0.0h2:0.0h3:1.0
* D: 0.10h1:0.0h2:0.0h3:1.0
* D: 0.12h1:0.0h2:0.0h3:1.0
* D: 0.14h1:0.0h2:0.0h3:1.0
* D: 0.16h1:0.0h2:0.0h3:1.0
* D: 0.18h1:0.0h2:0.0h3:1.0

[DISSOLUTION TEST=1 DEPTH=0.18]
* A:1.0h4:1.0
* A:2.0h4:1.0
* A:3.0h4:1.0
* A:4.0h4:1.0
* A:5.0h4:1.0
* A:6.0h4:1.0
* A:7.0h4:1.0
* A:8.0h4:1.0
* A:9.0h4:1.0
* A:10.0h4:1.0
* A:11.0h4:1.0
* A:12.0h4:1.0
A:13.04:377.0!
A:14.04:376.0!
A:15.04:376.0!
A:16.04:376.0!
A:17.04:376.0!
[DISSIPATION TEST END]

D:0.24:1.00:0.00:4.376.0!
D:0.24:1.00:0.00:4.376.0!

[ZERO OFFSETS AFTER TESTING] ④
<TIP> ④ 8.230
<LOCAL FRICTION> = -0.006
<PORR PRESSURE> ④ 2.000
[RND]

1: Data to be entered in the office or on the job; is job related
2: Data is known by user selection
3: Data to be entered in the office or on location; is test related
4: Fixed data for registration

④ General data:
When data is processed, additional data is added to existing data. The nature of this data is dependant on the user, no special rules apply. You recognise general data by <GORILLA!™ header> which proceeds them, indicating that this data come from GORILLA!™

② Calibration data and zero values of the parameters before the start of the test:
Calibration data of the different cones is stored in the datafile. They are used to scale test data. Before you start a test you are going to measure the zero values of the parameters. Make corrections where necessary. The test data in the next part has already been corrected with the calibration factors and the zero value. Parameters are not mentioned when they are not measured.
Test data

Storage of test data happens by using characters that have a special meaning. Because of this simple structure it is easy to read data into other processing programs. Please find below the characters and their meanings:

* is followed by a triggerbase (D=depth; A=time)
: is followed by a test value expressed in engineering units
# is followed by a channel number
! end of a complete scan

The following registration is interpreted as follows:

e.g.: D=0:3231:0.5#3:0.02#4:0.01

Registration happens on depth base (D), the value of the depth base=0.22 m
The value of channel 1=0.5
The value of channel 2=0.02
The value of channel 4=0.0

The following definitions are valid for the channel numbers:

1=tip resistance [MPa]
2=local friction [MPa]
3=pore pressure tip [kPa]
4=pore pressure shoulder [kPa]
5=pore pressure sleeve [kPa]
6=inclination [°]
7=total friction [kN]
8=total force [kN]
9=speed [m/s]
10=conductance [mS]
11=pH
12=redox [mV]
13=temperature [°C]
14=shear left [V]
15=shear right [V]
16=compression [V]
17=friction ratio [%]
18=conductivity [mS/m]

If you perform a dissipation test with a pore pressure cone, this is mentioned in the data. The number of the dissipation test is displayed as well as the depth at which the test was performed:

(DISSIMINATION TEST =1 DEPTH=39.04)

These data are followed by the registered values.

When using a cone, not all parameters are measured at one depth; in order to accomplish this all sensors should be located at the same spot on the cone. A compensation value can be given for each parameter. This value indicates the distance of the sensor with respect to the point where the depth registration takes place (tip resistance). The test data is corrected and stored in the data file.
Zero values of the parameters after the test is finished:
After the test, the cone is taken above ground level and the parameter values are once again registered unloaded. A large deviation in the zero-value at the start and the one at the end of the test may indicate damage to the cone. When a certain maximum permitted deviation in the values is exceeded, this is displayed on your screen.
2 Data flow GORILLA!®

To avoid problems with full floppy disks, full hard disks and data that cannot be found back, please follow the following work order.
The 3rd generation measuring system (touch screen) uses a "solid-state" disk of 8 Mb. Both the software and the data files are stored on this disk (D:).

If you use GORILLA!® together with a 2nd generation measuring system and a separate computer, we advise that you use the indicated directory structure. In most cases this structure will be the same as the structure used in the office.
The same directory structure is used for the office computer and for the computer in the truck.

Office:
C:\APB\GORILLA!
C:\APB\DATA
C:\APB\CONVERT

Truck (3rd generation):
D:\APB\GORILLA!
D:\APB\DATA
D:\APB\ARCHIVE

Truck (2nd generation):
C:\APB\GORILLA!
C:\APB\DATA
C:\APB\ARCHIVE

It is important to know that GORILLA!® in the office acts as the main station with respect to data storage and data processing.

2.1 DATA FLOW IN THE OFFICE
It is clear that data require a careful treatment. This is the reason that a number of functions involving data are performed in the office.

2.1.1 Job storage
The jobs to be done by the operator are created in the office. You use "STORE" from the DISK menu to copy the still empty files to the floppy disk.

The floppy disk in the A: drive will be automatically erased completely first before you copy the job data from C:\APB\DATA to A:

In addition to the empty job files also the cone data file (CONE.SDAT) is copied to the "job floppy", so that the operator always has the most recent cone list at his disposal.
2.1.2 test data read-in
After the jobs have all or partly been done, their data is copied from the "job floppy" to the data directory on the hard disk (C:\APB\DATA). To do this you use "LOAD" from the DISK menu. The data from the floppy are now added to the data already present on the disk.

2.1.3 new cone
With the acquisition of a new cone, we supply you, in addition to the datasheet, with a floppy disk containing the calibration data of the cone. By choosing "UPDATE" from the DISK menu, the new cone data is added to the existing list (C:\APB\DATA\CONES.DAT).

It is essential that these new cone data are entered in the office, as the "job floppy" must always contain an updated cone list (2.1.1).

2.1.4 calibrated cone
If A.P. van den Berg has calibrated a cone anew, the old cone data on the cone list must be replaced by the new data. This is done by using "UPDATE" from the DISK menu.

2.2 DATA FLOW ON THE TRUCK
Before performing a penetration test the operator must copy the "job floppy" that was prepared in the office. The still empty jobs are copied from the floppy disk to the data directory (D:\APB\DATA).

The test data present in the data directory will be automatically erased completely first before the new job data are copied from the floppy disk.

Once the data from the "job floppy" have been copied to the data directory, the floppy disk will be automatically erased completely. However, the cone list (CONES.DAT) remains on the floppy disk.

During a penetration test, data is stored on the solid state disk (D:). After the test the data is copied from the solid state disk (D:\APB\DATA) to the external drive (A:).
In addition the test is copied in compressed state from the data directory to the archive directory (D:\APB\ARCHIVE).
After a penetration test the test data is available in normal format (on floppy disk (A:) and on the solid state disk) and in compressed format.

As to the data in the archive directory: a minimum of 2000 m of test data (4 parameters) is stored. After some time the least recent data is automatically removed from the directory.

The compressed files are only available in DOS. They have the same name as the source file.

2.2.1 creating jobs on the truck

Not only in the office but also on the truck jobs can be created. They are stored in the data directory. The operator is now himself responsible for deleting the number of tests in the data directory (D:\APB\DATA) as well as on the "job floppy".

Test data that are no longer relevant can be removed by using "DELETE" from the JOB menu.

We advise that the operator always work with an empty floppy disk. In the office, after copying the data, the floppy disk is of course erased. This can also be done on the truck, in DOS.
3 Screen definitions GORILLAI®

A number of standard screen layouts have been defined for a clear display of the test data during a penetration test and have been delivered with the GORILLAI® software.

It is possible to adapt the existing definitions and/or to add new ones. These changes can be made by using an ASCII (DOS) editor.

The definitions are part of the file SCREENS.INI. We will discuss the structure using the example below.

```
[SCREENS]
<Screen 1>
<<Display LL>>
size=small
top channel=TIP
top scale index=1
bottom channel=none
bottom scale index=1
trigger base=D
base units per division=1
<<Display LL>>
size=small
top channel=LOCAL, FRICTION
top scale index=1
bottom channel=none
bottom scale index=1
trigger base=D
base units per division=1
<<Display NI>>
size=small
top channel=FRICTION HATIO
top scale index=1
bottom channel=none
bottom scale index=1
trigger base=D
base units per division=1
<<Display RI>>
size=small
top channel=INCLINATION
top scale index=1
bottom channel=none
bottom scale index=1
trigger base=D
base units per division=1
```
A screen consists of a number of individual displays (sub-screens). A screen definition can comprise a maximum of 4 displays. Each display can show a maximum of 2 parameters; one is shown with the description at the top and one with the description at the bottom of the screen. The triggerbase may be different per display. A display can have 4 different dimensions.

Several screen definitions can be part of the screen definition file (SCREENS.INI). The individual screens are indicated by placing the name of the screen definition in <> (e.g.: <screen 1>). Displays are indicated by placing the name of the display in << >>. Underneath the displays a number of keywords are mentioned. These keywords are followed by the sign =, which in its turn is followed by certain (fixed) texts. Below we explain the keywords and their meaning further.

Screen definitions
A screen consists of 20 horizontal divisions and 10 vertical divisions. These 20 divisions are distributed over the different displays. The user gives a name to the screen definition.

The name of the screen definition is used when you are referred to a screen definition while using a cone.

Displays
There is a maximum of 4 displays (sub-screens) defined per screen definition. Each display has a position and a certain size. The positions are fixed and are indicated as follows:

- display LL (left from left, starts at horizontal division position 0)
- display L  (left, starts at horizontal division position 5)
- display ML (middle, starts at horizontal division position 10)
- display R  (right, starts at horizontal division position 15)

Keywords
The fields belonging to a display are given below:

SIZE=
A display comes with four different widths. With these widths you can compose the matching screen. You can couple the following definitions to SIZE:

- small (5 horizontal divisions)
- medium (10 horizontal divisions)
- large (15 horizontal divisions)
- xlarge (20 horizontal divisions)
The total of divisions indicated by the individual display definitions can never be higher than 20.

E.g.: display LL=medium, display MI=small, display R=small.
Display L is left out because this position is overlapped.

TOP CHANNEL=
A definition created in CHANNELS.INI (see chapter 4) is coupled to TOP CHANNEL. This definition applies to the display of a parameter.

It is necessary that the text entered after TOP CHANNEL= exactly agrees with the channels that were earlier defined in CHANNELS.INI. The use of lower case and higher case however makes no difference.

If you want the top of a display to show the value of the tip, you enter this as follows:

TOP CHANNEL=TIP

TOP SCALE INDEX
The parameter definition in CHANNELS.INI comprises the general definition as well as a number of definitions relating to the units and the graduated scale. The index of one of these definitions is entered after TOP SCALE INDEX.

BOTTOM CHANNEL=
A definition created in CHANNELS.INI (see chapter 4) is coupled to BOTTOM CHANNEL. This definition applies to the display of a parameter.

It is necessary that the text entered after BOTTOM CHANNEL= exactly agrees with the channels that were earlier defined in CHANNELS.INI. The use of lower case and higher case however makes no difference.

If you want the bottom of a display to show the value of the friction, you enter this as follows:

BOTTOM CHANNEL=LOCAL FRICTION

BOTTOM SCALE INDEX=
The parameter definition in CHANNELS.INI comprises the general definition as
well as a number of definitions relating to the unities and the graduated scale. The index of one of these definitions is entered after BOTTOM SCALE INDEX.

**TRIGGER BASE=**
You have a choice of two different triggerbases for the display of the values: depth base and time base. For these bases it is valid that:

- D is depth base in meters
- A is time base in seconds

In order to display the data on depth base you must enter:

**TRIGGER BASE=D**

**BASE UNITS PER DIVISION=**
In a display a total of 10 divisions can be shown for a triggerbase. The value of the selected base per division is entered after BASE UNITS PER DIVISION=.

**e.g.:** For a display of maximum 5 meter on the complete screen, you enter:

**BASE UNITS PER DIVISION=0.5**
4  Parameter definitions GORILLA!

You can find the parameters that can be measured by GORILLA!® in the definition file CHANNELS.INI. This file holds the general settings of the parameter as well as information regarding the further processing of the test data.

A number of settings are standard and cannot be changed. This will be indicated where applicable.

The first line of the file CHANNELS.INI is always [CHANNELS].

Below we will treat the settings belonging to the parameter definition of the TIP:

[CHANNELS]
<TIP>
channel #=1
paramKey=TIP
log=TIP RESISTANCE
 linestyle=solid
 color=lightred
 log to file=on
 log to printer=on
 recorder channel=0
 recorder scale=1
 recorder offset=0
 alarm level=50
 alarm active=off
 shutdown=off
 <<scale definition 1>>
 engineering units=[MPa]
 decimal position=2
 engineering unit per division=2.5
 first value on X-axis=0
 stroke to engineering factor=1
 <<scale definition 2>>
 engineering units=
 ....
 ....

The name of the parameter appears in < >.
CHANNEL #=
At this prompt you type the number as defined in the list below:

1 = tip resistance [MPa]  
2 = local friction [MPa]  
3 = pore pressure tip [kPa]  
4 = pore pressure shoulder [kPa]  
5 = pore pressure sleeve [kPa]  
6 = inclination [*]  
7 = total friction [kN]  
8 = total force [kN]  
9 = speed [m/s]  
10 = conductance [mS]  
11 = pH  
12 = redox [mV]  
13 = temperature [°C]  
17 = shear left [V]  
18 = shear right [V]  
19 = compression [V]  
20 = friction ratio [%]  
21 = conductivity [mS/m]

PARAMKEY=
This field connects the screen definition files and the cone definition files with the help of cross-references.

"The text that follows PARAMKEY= may not be changed as the connection with other files is dependent on this text."

TAG=
Is followed by the description of the parameter as displayed on the screen.

LINESTYLE=
Here you enter the type of line in which you would like the parameter to be displayed. You can choose from 4 fixed styles:

- solid
- dotted
- center
- dashed

"You must enter the text for LINESTYLE= exactly as is indicated above."
COLOR=
Here you enter the color you wish for your parameter. You can choose from 16 colors:

black  white
darkgray lightgray
blue  lightblue
green  lightgreen
cyan  lightcyan
red  lightred
magenta  lightmagenta
brown  yellow

You must enter the text for COLOR= exactly as is indicated above.

LOG TO FILE=
Normally all parameters will be stored. In that case LOG TO FILE= should be followed by on. If a certain parameter does not need to be stored, you enter off.

LOG TO PRINTER=
Here you enter on for the on-line registration on a printer of the test data. If you enter off, the test data of the relevant parameter will not be printed.

In order to have the test data of the parameters in print on-line, you need to select the option HARDCOPY from the main menu of GORILLA! followed by the function ON-LINE. During the test, the screen displays which parameters are being printed.

RECORDER CHANNEL=
This option is followed by 0 and cannot be changed.

RECORDER SCALE=
This option is followed by 1 and cannot be changed.

RECORDER OFFSET=
This option is followed by 0 and cannot be changed.
ALARM LEVEL=
It is possible to set an alarm level for every parameter. If the level is exceeded, an acoustic signal is given or the hydraulic system shuts off, depending on the further setting. The alarm level is shown as a dotted vertical line on the display in question.

ALARM ACTIVE=
However you have to activate the alarm level by typing on at the ALARM ACTIVE= prompt.
Now, as soon as the alarm level is exceeded, an acoustic signal is given. If you type off, no signal is given and no dotted line is displayed.

SHUTDOWN=
If, in addition, you want an automatic shut-off of the hydraulic system as soon as the alarm level is exceeded, you type on at the SHUTDOWN= prompt. If you don't want an automatic shut-off, you type off.

In order to be able to shut-off the hydraulic system, the system must be equipped with the appropriate hardware.

For one parameter several definitions can be entered that relate to the units and the graduated scale on screen.

These factors do not affect the value as stored in the data file.
The different definitions are selected by indicating the number of the definition as follows:

<<scale definition 1>>

Please find below the fields within this definition.

ENGINEERING UNIT=
Please enter the text displayed, exactly as shown.

DECIMAL POSITION=
Please enter the position of the decimal point.
ENGINEERING UNIT PER DIVISION=
For a correct graduated scale you type in the desired value. It is best to choose these definitions in such a way that they link up with different display dimensions. In order to reach a full range of 50 MPa using a display of 5 divisions (small), a value of 10 should be used.
If you also want a full range of 50 MPa using a display of 10 divisions (medium), you create a second scale definition to set the ENGINEERING UNIT PER DIVISIONS to 5.

FIRST VALUE ON X-AXIS=
For parameters that can have a negative value you enter a negative starting value. For those parameters that only reach positive values, you enter 0.

STREAM TO ENGINEERING FACTOR=
This option shows 1 and cannot be changed.
Cone definitions GORILLA!

By using GORILLA!® it is no longer necessary to calibrate the cones before a test. The software corrects the measured data. As the calibration data is different for each cone, these data must be available. You can find these data in the file CONES.DAT. By selecting a certain cone for a test, its calibration factors are applied to the test data. CONES.DAT contains also other information. Below we explain the settings important to the user.

A number of settings are standard and, where indicated, should not be changed.

The first line of the file CONES.DAT is always [CONES].

By using the example of an ELCF cone we treat the settings belonging to this cone below:

[CONES]
<cone 1>
cone number=  
cone type=  
calibration date=  
screen nx=  
comment=  
<<TIP>>
ccl init=  
ccl input=  
ccl output=  
scaling factor=  
scaling offset=  
auto zero=  
zero offset after testing=  
maximum zero offset=  
exitation=  
sensor displacement (cm)=  
<<LOCAL FRICITION>>
ccl init=  
ccl input=  
ccl output=  
scaling factor=  
scaling offset=  
auto zero=  
zero offset after testing=  
maximum zero offset=  
exitation=  
sensor displacement (cm)=  


The file contains general information regarding the cone as well as information about the parameters that can be measured using the cone. The general part follows directly under <cone 1>. In this part you can find several settings related to the type of cone and not to the parameters as such.

Information relating to the parameters follows after the name of the parameter in << >>.
Below we will first describe the general information, then the specific parameter information:

CONE NUMBER=
Is followed by the serial number of the cone.

CONE TYPE=
Is followed by the type of cone.

CALIBRATION DATE=
Is followed by the most recent date that the cone was calibrated.

SCREEN NR=
When you select a cone, you also select a screen structure to go with it. You type the desired screen division at the SCREEN NR= prompt, making sure to use the exact text the way it is indicated in the file SCREENS.INI, whereby upper - or lower case has no influence.

E.g. : SCREEN NR=screen 1

COMMENT=
You can give a description with every cone. Normally you type here the parameter ranges, but different text is also possible, of course.

The settings relating to the general information of the cone are followed by the definitions of the individual parameters measured with the cone.

The name of the parameter is typed inside << >> and must be copied exactly from the parameter definition file CHANNELS.INI. (It does not matter if you use upper - or lower case). The definition of tip resistance is entered in the file as follows:
<<TIP>>
CCI INIT=
With the values input here several settings of the hardware can be changed.

These settings are standard and should not be changed.

CCI INPUT=
The value refers to the hardware setting of the measuring system.

This value should not be changed.

CCI OUTPUT=
This value refers to the hardware setting of the measuring system.

This setting should not be changed.

SCALING FACTOR=
You enter the scaling factor necessary to correctly scale in the test result.

SCALING OFFSET=
You type in the fixed factor needed to correct the test result, if necessary.

AUTO ZERO=
If followed by on, the test results are corrected with the measured offset values. If followed by off, the test results are not corrected. See also the standard settings in the file DEFAULT.INI.

ZERO OFFSET AFTER TESTING=
Here you can store the zero-value at the end of the test.

MAXIMUM ZERO OFFSET=
Here you enter the maximum permissible offset value. If this value is exceeded when you measure the zero-values, you will see this on the relevant screen.

EXITATION=
This value refers to the hardware of the measuring system.

This setting should not be changed.
SENSOR DISPLACEMENT (CM)=
This value indicates the distance between the sensor and the spot where depth registration takes place (tip resistance).
5 Standard settings GORILLA!

A number of standard settings are stored in the file DEFAULT.INI. We create this file before delivery and normally this file does not need to be changed. However it is possible for the user to change the settings. Only those settings that can be changed by the user will be discussed below.

LANGUAGE=
All text, messages and error messages can be displayed in several languages. You can enter the text below at the LANGUAGE= prompt:
- ENG, UK, USA (English)
- GER, DR (German)
- FRA (French)
- NED, NL (Dutch)

DEVICE=
Here you indicate whether GORILLA!® is used on site or in the office.
- SLAVE (truck)
- HOST (office)

PRINTER PORT=
Indicates where your output is sent, when you use the on-line print function.
- LPT1 (parallel port 1)
- LPT2 (parallel port 2)
- COM1 (serial port 1)
- COM2 (serial port 2)

PROGRAM PATH=
As we have mentioned, the software is organized using a fixed directory structure. If necessary this structure can be modified. You type at the PROGRAM PATH= prompt the new directory where the program is stored.

DATA PATH=
The test results are stored in the directory displayed at the DATA PATH= prompt.

DRIVE PATH=
When the test is finished, the data is copied from the data directory to an external drive. This drive is indicated here, though it is possible to type in a directory.
CONVERSION DESTINATION PATH=
The directory indicated here holds the results of the conversion if you have used the APB-conversion program.

CONVERSION PROGRAM=
Is followed by the name of the APB-conversion program.

ARCHIVE PATH=
Is followed by the directory that holds the compressed data.

ARCHIVE COMMAND LINE=
The program used for test data compression is fixed and should not be changed.

MINIMUM DISK SPACE (MB)=
GORILLA® checks continuously how much disk space is available to store test data. For storage in the DATA directory a fixed storage capacity is reserved. The size of this capacity is indicated at the MINIMUM DISK SPACE= prompt. The remainder of the disk capacity is used to keep the records of the compressed test data. If the minimum disk capacity is exceeded, the oldest files from the ARCHIVE directory are removed.

MINIMUM ARCHIVE SIZE (MB)=
Here you find the capacity of the disk reserved for data to be stored in the ARCHIVE directory. The bigger this space, the more files can be kept.

PRINTER BASE INTERVAL (CM)=
The test results are registered every 2 cm. If you use the on-line print function, it is possible to indicate a different interval for the printer to print the test results. Usually this interval is 10 cm.

! The value typed at the PRINTER BASE INTERVAL (CM)= prompt must be a multiple of 2.
DISSIPATION TEST SCREEN LAYOUT=
Please type in the screen description exactly the way it is defined in SCREENS.INI, so that when testing with a pore pressure cone to do a dissipation test GORILLA\textsuperscript{\textregistered} automatically changes to the matching screen structure.

ENVIRO TIMEBASE SCREEN LAYOUT=
Please type in the screen description exactly the way it is defined in SCREENS.INI, so that when testing with an envirocone to do a test on time base GORILLA\textsuperscript{\textregistered} automatically changes to the matching screen structure.

SEISMIC TEST SCREEN LAYOUT=
Please type in the screen description exactly the way it is defined in SCREENS.INI, so that when testing with a seismic cone to do a seismic test GORILLA\textsuperscript{\textregistered} automatically changes to the matching screen structure.

ZERO SCAN=
Here is indicated whether the zero values should be measured before and after a test.
- on (zero values to be measured)
- off (measurement of the zero values can be skipped)

ERROR SOUND=
If followed by on, an error message on screen is accompanied by an alarm. If followed by off, only the error message is displayed.

ALLOW EDIT TEST AT START=
If followed by on (or yes), this means that this function is active. Before a test is started, GORILLA\textsuperscript{\textregistered} displays the additional information of this test. It is possible to change, or add to, this information. If followed by off this option is skipped. In that case the header can be changed by using the JOB(s) menu from the main menu.

DA PORT LOG=
Is followed by off, because normally this option is not used. However, in certain emergency cases you may wish to store all data, as they are measured by the data acquisition system, in a separate file. So if DA PORT LOG= is followed by on, the data is stored in this file {DA_LOG}.GOR.
AIM BASE ADDRESS=
Only in those cases where a seismic cone is being used, is this option important. Its setting may only be changed in consultation with A.P. van den Berg.
C.1 INTRODUCTION

An extensive experimental program was conducted by Russell (2004) to characterize saturated and unsaturated Sydney sand. Certain test results of Russell (2004) including soil water characteristic curves (SWCCs), isotropic compression tests on saturated specimens, drained and undrained compression shear tests on saturated specimens and constant suction and constant moisture content compression shear tests on unsaturated specimens are represented here. The SWCCs are used to determine the required air entry or air expulsion values in definition of effective stress in unsaturated Sydney sand. The triaxial test results presented are used in describing stress strain behaviour of Sydney sand and calibration of the constitutive model adopted (chapter 5).
C.2 SOIL-WATER CHARACTERISTIC CURVES
Russell (2004) conducted a number of filter paper tests and pressure plate tests to investigate the soil water characteristic (SWCC) for Sydney sand. The SWCCs were determined for both drying and wetting paths of the specimens. The results are represented here.

The filter paper and pressure plate test results are presented in Figures C.1 to C.3 in the $w \sim \log s$ and $\log w \sim \log s$ planes for specimens prepared at the three different initial void ratios. The solid symbols represent a drying path and the open symbols represent a wetting path. It can be seen that the filter paper test results are similar to the pressure plate test results.

C.3 MECHANICAL BEHAVIOUR OBSERVED IN TRIAXIAL TESTS
Numerous triaxial tests were performed on saturated and unsaturated Sydney sand by Russell (2004). These included isotropic compression tests on saturated specimens at low and high stresses, drained and undrained compression shear tests on saturated specimens at low and high stresses; and constant suction and constant moisture content compression shear tests on unsaturated specimens at low stresses. Only the tests conducted at low confining stresses have been represented here.

C.3.1 Saturated Isotropic Compression Tests
The results of the isotropic compression tests are presented in the $v \sim \log p'$ plane in Figure C.4. The tests were conducted on saturated specimens of initial specific volumes $v_0 = 1.774, 1.738, 1.709$ and 1.690; and are referred to using the symbols ISO-L, ISO-M, ISO-D and ISO-VD, respectively.

C.3.2 Saturated Drained Compression Shear Tests
The results of saturated drained triaxial compression shear tests are presented in Figure C.5 in the $q \sim \varepsilon_q$ and $\varepsilon_p \sim \varepsilon_q$ planes. The tests conducted at constant $\sigma'_3$ values of 50kPa, 115kPa, 157kPa, 242kPa and 301kPa are referred to using the symbols 50D,
115D, 157D, 242D and 301D; and after isotropic compression had initial specific volumes of \( v_0 = 1.677, 1.685, 1.670, 1.735 \) and 1.730, respectively.

### C.3.3 Saturated Undrained Compression Shear Tests

The results of saturated undrained compression shear tests are presented in Figure C.6 in the \( q \sim p' \) and \( q \sim \varepsilon_q \) planes. The tests conducted at constant \( \sigma_3 \) values of 300kPa on specimens that, after isotropic compression, had initial specific volumes of \( v_0 = 1.917, 1.830 \) and 1.721, are referred to using the symbols 300U-L, 300U-M and 300U-D, respectively.

### C.3.4 Unsaturated Compression Shear Tests

The results of unsaturated drained triaxial compression shear tests are presented in Figures C.7 to C.9 in the \( q \sim p' \) and \( \varepsilon_p \sim \varepsilon_q \) planes. The tests conducted at constant \( \sigma_{3n} \) values of 50kPa and 102kPa, each with a constant \( s \) value of 51kPa, with initial specific volumes of \( v_0 = 1.770 \) and 1.658, are referred to using the symbols 5050L-D and 10050D-D, respectively. The tests conducted at constant \( \sigma_{3n} \) values of 50kPa and 100kPa, each with a constant \( s \) value of 100kPa, with initial specific volumes of \( v_0 = 1.763 \) and 1.687, are referred to using the symbols 50100L-D and 100100D-D, respectively. The tests conducted at constant \( \sigma_{3n} \) values of 51kPa and 100.5kPa, with constant \( s \) values of 198kPa and 199.5kPa, with initial specific volumes of \( v_0 = 1.780 \) and 1.697, are referred to using the symbols 50200L-D and 100200D-D, respectively.

The results of unsaturated undrained triaxial compression shear tests are presented in Figures C.10 to C.12 in the \( q \sim \varepsilon_q \) and \( q \sim \varepsilon_q \) planes. The tests conducted at constant \( \sigma_{3n} \) values of 50kPa and 100kPa, each with an initial \( s \) value of 50kPa, with initial specific volumes of \( v_0 = 1.771 \) and 1.676, are referred to using the symbols 5050L-U and 10050D-U, respectively. The tests conducted at constant \( \sigma_{3n} \) values of 49kPa and 98kPa, with initial \( s \) values of 98kPa and 100kPa, with initial specific volumes of \( v_0 = 1.777 \) and 1.674, are referred to using the symbols 50100L-U and 100100D-U, respectively. The tests conducted at constant \( \sigma_{3n} \) values of 49kPa and 100kPa, with initial \( s \) values of 201kPa and 199kPa, with initial specific volumes of \( v_0 = 1.747 \) and 1.688, are referred to using the symbols 50200L-U and 100200D-U, respectively.

C-3
C.3.5 Discussion

In the isotropic compression tests a gradual increase in the compressibility and therefore a reduction in the stiffness has been observed with increasing logarithm of stress, rather than a distinct break as is typically observed for clays.

The saturated drained compression shear test results are typical of those for quartz sand. Specifically, the results for tests conducted on dense specimens at low stresses exhibit hardening up to a peak in the shear resistance, accompanied by initial volumetric contraction followed by volumetric expansion. Softening towards a critical state is observed after reaching the peak and is accompanied by volumetric expansion. In the saturated undrained compression shear tests, the results for initially loose specimens exhibit a peak in their shear resistance followed by a gradual reduction. The results for initially slightly dense specimens exhibit a peak in the shear resistance, followed by a reduction, and then an increase. The results for denser specimens exhibit a continual increase in shear resistance from the start of the test.

The compression shear test results for the unsaturated specimens, all of which were initially dense, appear to be similar for both drained and undrained conditions. The behavior, in general, is the same as that observed for the drained tests conducted on saturated and dense specimens at low stresses.
Figure C.1 SWCCs for Sydney sand with $e = 0.68 \pm 0.013$ in the $w \sim \log s$ plane (a) and the $\log w \sim \log s$ plane (b) (after Russell, 2004).
Figure C.2 SWCCs for Sydney sand with $e = 0.73 \pm 0.015$ in the $w \sim \text{logs}$ plane (a) and the $\log w \sim \text{logs}$ plane (b) (after Russell, 2004).
Figure C.3 SWCCs for Sydney sand with $e = 0.78 \pm 0.018$ in the $w \sim \log s$ plane (a) and the $\log w \sim \log s$ plane (b) (after Russell, 2004).
Figure C.4 Isotropic compression test results for Sydney sand presented in the $\nu \sim \log p'$ plane, conducted on saturated specimens and initial conditions of $v_0 = 1.774$ (ISO-L), $v_0 = 1.738$ (ISO-M), $v_0 = 1.709$ (ISO-D) and $v_0 = 1.690$ (ISO-VD) (after Russell, 2004).
Figure C.5 Saturated drained triaxial compression shear test results for Sydney sand presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_q \sim \varepsilon_q$ (b) planes, with constant $\sigma'_3$ values of 50 kPa (50D), 115 kPa (115D), 157 kPa (157D), 242 kPa (242D) and 301 kPa (301D), and initial specific volumes of $v_0 = 1.677, 1.685, 1.670, 1.735$ and 1.730, respectively (after Russell, 2004).
Figure C.6 Saturated undrained triaxial compression shear test results for Sydney sand presented in the $q \sim p'$ (a) and $q \sim \varepsilon_q$ (b) planes, each with constant $\sigma_3$ values of 300 kPa, and initial specific volumes of $v_0 = 1.917$ (300U-L), 1.830 (300U-M) and 1.721 (300U-D), respectively (after Russell, 2004).
Figure C.7 Unsaturated drained triaxial compression shear test results for Sydney sand presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant $\sigma_{mn}$ values of 50 kPa (5050L-D) and 102 kPa (10050D-D), each with a constant $s$ value of 51 kPa, and initial specific volumes of $v_0 = 1.770$ and 1.658, respectively (after Russell, 2004).
Figure C.8 Unsaturated drained triaxial compression shear test results for Sydney sand presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant $\sigma_{mn}$ values of 50 kPa (50100L-D) and 100 kPa (100100D-D), each with a constant $s$ value of 100 kPa, and initial specific volumes of $v_0 = 1.763$ and 1.687, respectively (after Russell, 2004).
Figure C.9 Unsaturated drained triaxial compression shear test results for Sydney sand presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant $\sigma_{3n}$ values of 51 kPa (50200L-D) and 100.5 kPa (100200D-D), constant $s$ values of 198 kPa and 199.5 kPa, and initial specific volumes of $v_0 = 1.780$ and 1.697, respectively (after Russell, 2004).
Figure C.10 Unsaturated undrained triaxial compression shear test results for Sydney sand presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant $\sigma_{3n}$ values of 50 kPa (5050L-U) and 100 kPa (10050D-U), each with an initial $s$ value of 50 kPa, and initial specific volumes of $v_0 = 1.771$ and 1.676, respectively (after Russell, 2004).
Figure C.11 Unsaturated undrained triaxial compression shear test results for Sydney sand presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant $\sigma_{3u}$ values of 49 kPa (50100L-U) and 98 kPa (100100D-U), initial $s$ values of 98 kPa and 100 kPa, and initial specific volumes of $v_0 = 1.777$ and 1.674, respectively (after Russell, 2004).
Figure C.12 Unsaturated undrained triaxial compression shear test results for Sydney sand presented in the $q \sim \varepsilon_q$ (a) and $\varepsilon_p \sim \varepsilon_q$ (b) planes, with constant $\sigma_{3m}$ values of 49 kPa (50200L-U) and 100 kPa (100200D-U), initial $s$ values of 201 kPa and 199 kPa, and initial specific volumes of $v_0 = 1.747$ and 1.688, respectively (after Russell, 2004).
APPENDIX D

POST TEST MOISTURE CONTENT DISTRIBUTION

Post test core samples, using a hand operated auger of 50 mm diameter, were taken from different locations of the chamber specimens to investigate the moisture distribution. Examples of moisture content values throughout the specimen are presented here.

Moisture content values throughout the 460 mm diameter and 840 mm high chamber specimen are tabulated in Tables D.1 to D.6. First column of these tables presents depth of the recovered sample and the second column presents the radial distance from centre of the chamber specimen.
Table D.1 Post test moisture distribution throughout the chamber specimen for the test UM100-200 (specimen with an initial relative density of \(D_r = 61\%\) subjected to isotropic net confining stress of 100 kPa and an initial controlled suction of 200 kPa).

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Radial distance (mm)</th>
<th>Moisture content (w) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>centre</td>
<td>1.7</td>
</tr>
<tr>
<td>200</td>
<td>100</td>
<td>1.6</td>
</tr>
<tr>
<td>200</td>
<td>180</td>
<td>1.6</td>
</tr>
<tr>
<td>300</td>
<td>centre</td>
<td>1.7</td>
</tr>
<tr>
<td>300</td>
<td>100</td>
<td>1.8</td>
</tr>
<tr>
<td>300</td>
<td>180</td>
<td>1.8</td>
</tr>
<tr>
<td>450</td>
<td>centre</td>
<td>1.7</td>
</tr>
<tr>
<td>450</td>
<td>100</td>
<td>1.9</td>
</tr>
<tr>
<td>450</td>
<td>180</td>
<td>1.9</td>
</tr>
<tr>
<td>600</td>
<td>centre</td>
<td>1.8</td>
</tr>
<tr>
<td>600</td>
<td>100</td>
<td>1.6</td>
</tr>
<tr>
<td>600</td>
<td>180</td>
<td>1.7</td>
</tr>
<tr>
<td>750</td>
<td>centre</td>
<td>1.9</td>
</tr>
<tr>
<td>750</td>
<td>100</td>
<td>1.9</td>
</tr>
<tr>
<td>750</td>
<td>180</td>
<td>1.7</td>
</tr>
</tbody>
</table>
Table D.2 Post test moisture distribution throughout the chamber specimen for the test UM50-200 (specimen with an initial relative density of $D_r = 61\%$ subjected to isotropic net confining stress of 50 kPa and an initial controlled suction of 200 kPa).

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Radial distance (mm)</th>
<th>Moisture content ($w$) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>centre</td>
<td>1.5</td>
</tr>
<tr>
<td>250</td>
<td>100</td>
<td>1.6</td>
</tr>
<tr>
<td>250</td>
<td>180</td>
<td>1.6</td>
</tr>
<tr>
<td>350</td>
<td>centre</td>
<td>1.7</td>
</tr>
<tr>
<td>350</td>
<td>100</td>
<td>1.8</td>
</tr>
<tr>
<td>350</td>
<td>180</td>
<td>1.7</td>
</tr>
<tr>
<td>500</td>
<td>centre</td>
<td>1.6</td>
</tr>
<tr>
<td>500</td>
<td>100</td>
<td>1.7</td>
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<tr>
<td>500</td>
<td>180</td>
<td>1.7</td>
</tr>
<tr>
<td>600</td>
<td>centre</td>
<td>1.7</td>
</tr>
<tr>
<td>600</td>
<td>100</td>
<td>1.8</td>
</tr>
<tr>
<td>600</td>
<td>180</td>
<td>1.7</td>
</tr>
</tbody>
</table>
Table D.3 Post test moisture distribution throughout the chamber specimen for the test UL50-200 (specimen with an initial relative density of $D_r = 33\%$ subjected to isotropic net confining stress of 50 kPa and an initial controlled suction of 200 kPa).

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Radial distance (mm)</th>
<th>Moisture content ($w$) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>centre</td>
<td>1.6</td>
</tr>
<tr>
<td>250</td>
<td>100</td>
<td>1.5</td>
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<tr>
<td>250</td>
<td>180</td>
<td>1.6</td>
</tr>
<tr>
<td>350</td>
<td>centre</td>
<td>1.6</td>
</tr>
<tr>
<td>350</td>
<td>100</td>
<td>1.6</td>
</tr>
<tr>
<td>350</td>
<td>180</td>
<td>1.7</td>
</tr>
<tr>
<td>500</td>
<td>centre</td>
<td>1.8</td>
</tr>
<tr>
<td>500</td>
<td>100</td>
<td>1.8</td>
</tr>
<tr>
<td>500</td>
<td>180</td>
<td>1.7</td>
</tr>
<tr>
<td>650</td>
<td>centre</td>
<td>1.8</td>
</tr>
<tr>
<td>650</td>
<td>100</td>
<td>1.8</td>
</tr>
<tr>
<td>650</td>
<td>180</td>
<td>1.8</td>
</tr>
</tbody>
</table>
Table D.4 Post test moisture distribution throughout the chamber specimen for the test UL25-50 (specimen with an initial relative density of $D_r = 33\%$ subjected to isotropic net confining stress of 25 kPa and an initial controlled suction of 50 kPa).

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Radial distance (mm)</th>
<th>Moisture content ($w$) (%)</th>
</tr>
</thead>
<tbody>
<tr>
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<td>centre</td>
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</tr>
<tr>
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<td>250</td>
<td>180</td>
<td>2.9</td>
</tr>
<tr>
<td>350</td>
<td>centre</td>
<td>2.9</td>
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<tr>
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<td>100</td>
<td>2.8</td>
</tr>
<tr>
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<tr>
<td>500</td>
<td>centre</td>
<td>3.0</td>
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<tr>
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<td>3.0</td>
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<tr>
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<td>180</td>
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</tr>
<tr>
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<td>centre</td>
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<tr>
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<tr>
<td>650</td>
<td>180</td>
<td>3.0</td>
</tr>
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</table>
Table D.5 Post test moisture distribution throughout the chamber specimen for the test UL50-25 (specimen with an initial relative density of $D_r = 33\%$ subjected to isotropic net confining stress of 50 kPa and an initial controlled suction of 25 kPa).

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Radial distance (mm)</th>
<th>Moisture content ($w$) (%)</th>
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</thead>
<tbody>
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<td>centre</td>
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<tr>
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<tr>
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<tr>
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<td>180</td>
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</table>
Table D.6 Post test moisture distribution throughout the chamber specimen for the test UM100-25 (specimen with an initial relative density of $D_r = 61\%$ subjected to isotropic net confining stress of 100 kPa and an initial controlled suction of 25 kPa).

<table>
<thead>
<tr>
<th>Depth (mm)</th>
<th>Radial distance (mm)</th>
<th>Moisture content ($w$) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>250</td>
<td>centre</td>
<td>4.6</td>
</tr>
<tr>
<td>250</td>
<td>100</td>
<td>4.5</td>
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<tr>
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<tr>
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<td>4.7</td>
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<tr>
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<td>4.8</td>
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<tr>
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<tr>
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